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POWER DIVISION

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CONTROL OF CRACKING IN CONCRETE GRAVITY DAMS

William R. Waugh,[1] F. ASCE and James A. Rhodes,[2] M. ASCE

ABSTRACT

The fundamental cause of temperature cracking in concrete gravity dams is the restraint which accompanies temperature changes. Paper traces evolution of crack control measures (cement factor, cement replacement material, concrete placing temperature, lift thickness, rate of placement, and insulation), and describes current Corps of Engineers practices and results.

INTRODUCTION

Analytically derived stress patterns in concrete gravity dams, based on the usual dead and live loads expected to exist, frequently resemble only vaguely stresses measured in the prototype structures. Visible evidences of the inadequacy of simple stress analyses are the cracks which develop in a structure that are not remotely related to theoretical dead and live load stresses. The principal cause of such cracking is volume changes accompanying temperature variations within the structure.

It is the purpose of this paper to present the methods used and the results obtained by the Corps of Engineers in reducing the magnitude, extent and amount of structurally significant temperature cracking in mass concrete structures. If such cracking can be avoided, it then follows that temperature induced stresses are not excessive and the design assumptions as to the unity of the structure are valid.

There are other potential causes of cracking which may be encountered in mass concrete construction, but these will not be discussed. Prominent among

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these is the cracking which may occur as a result of alkali-aggregate reaction.

The effects of this and other factors on crack development in a mass concrete structure can be minimized by adequate care in the selection of concrete materials and the exercise of proper practices during construction.

The cracks resulting from temperature changes which are of greatest concern are the vertical cracks parallel to the axis which if extensive enough can interrupt the continuity of a dam and unfavorably influence its structural integrity. Such cracks usually result from temperature changes which occur during construction and are evident on the exposed bulkhead faces and across the tops of monoliths. Initially such cracks may appear to be superficial or surface cracks of only shallow depth. Measurements of the depths of such cracks made soon after initial detection have encouraged what is believed to be an erroneous conclusion that such cracks will not completely dissect a monolith and therefore should be of no concern from a structural standpoint. Initially such cracks may penetrate to a depth of only a few feet but subsequent temperature changes and the influence of load may cause a crack to extend so that it eventually passes completely through a monolith from the foundation to the downstream face. For many years there was much speculation concerning this phenomenon and those who have attempted to discount the importance of control of cracking have contended that no such extension of cracks could or would occur. Until recent years no systematic exploration was undertaken for the purpose of positively determining the extent of such cracks; however, a detailed exploratory examination was undertaken of a concrete gravity dam by drilling NX core holes (3-inch diameter), and by photographing the interior of the holes with a borehole camera[1] to supplement the evidence obtained from the drilling operations. The exploration proved conclusively that cracks do sometimes extend laterally completely through a monolith from contraction joint to contraction joint and vertically from near the foundation to near the downstream face.

Cracks which may unfavorably influence structural integrity are of greatest concern; however, cracks which unfavorably influence water tightness, durability and appearance are also objectionable. Vertical cracks normal to the axis of a dam and horizontal cracks, both of which sometimes occur on upstream and downstream faces of dams, cause leakage, may adversely affect durability in severe climates and detract materially from the appearance of a dam.

Fundamental Cause of Temperature Cracking

The fundamental cause of temperature cracking is restraint. Two prominent restraint situations are common in mass concrete dams. One of these results when differences in temperature occur in a dam. The particular situation which causes cracking occurs when exposed portions of a dam are severely cooled when subjected to low ambient temperatures while the adjacent interior portions are at relatively high temperatures. The warm interior concrete restrains the cool exterior concrete from contracting freely and when tensile stress in excess of tensile strength develops, a crack occurs.

The other prominent restraint situation occurs when concrete is cast on a relatively rigid rock foundation or on relatively rigid old concrete which has cooled. As the temperature of concrete rises it is restrained from freely

expanding by the rock or old concrete and tends to develop compressive stresses. Measurements of structural behavior of mass concrete dams[4] have indicated however that very little compressive stress develops as a result of the initial rise in temperature because of low modulus of elasticity and the relatively large amount of creep which occurs. So a relatively minor decline in temperature relieves the small compressive stress and all further declines in temperature against the restraining influence of the foundation sets up tensile stresses.

It is of interest to speculate that there is a distinct possibility that cracks first caused on bulkhead faces by restraint due to sharp differences in temperature between interior and exterior portions of a monolith may be extended entirely through a monolith by subsequent effects of foundation restraint. If on the other hand, no cracks exist on a bulkhead face a considerable drop in temperature can occur, over an extended period of time (a matter of years), against foundation restraint without causing a crack.[2]

Factors Influencing Temperature Changes

The factors which have the greatest influence may be summarized as follows:

1. Type of cementing material used.
2. Quantity of cementing material used.
3. Construction program and procedures.
4. Artificial Treatments.
5. Thermal properties.
6. Climatic conditions.

The first four of these are subject to regulation within limits and through such regulation provide the means for control of temperatures. The heat generating characteristics and amount of cementing material used have a very great influence on the magnitude of the temperature changes which occur.

The construction program and procedures influence to a marked degree the problem of temperature control. Through proper planning and regulation of the construction program, circumstances which are conducive to cracking can be largely avoided. The ideal construction program as far as temperature control is concerned is one in which (a) all the monoliths of a dam are brought up uniformly, with the successive lifts in each monolith cast at regular intervals, (b) the intervals between casting of successive lifts are kept at the optimum for the type of control utilized, and (c) the difference in elevation between adjacent monoliths is kept at a practical minimum. This ideal is rarely approached because usually for practical reasons the work has to be prosecuted in stages wherein provisions are made for diversion of the stream. This results in interruptions in the work in some monoliths while work in other monoliths is being prosecuted (Fig. 1). Some monoliths are left low over which the stream is temporarily diverted while adjacent monoliths are carried up to heights considerably above the level of the low diversion blocks. The resulting high bulkhead faces if exposed during extremely cold weather will almost invariably crack. The top surfaces of monoliths in which the progress of the work is interrupted for a long period of time are also vulnerable to cracking under such circumstances. The ideal in construction programming is most nearly approached when the stream can be completely

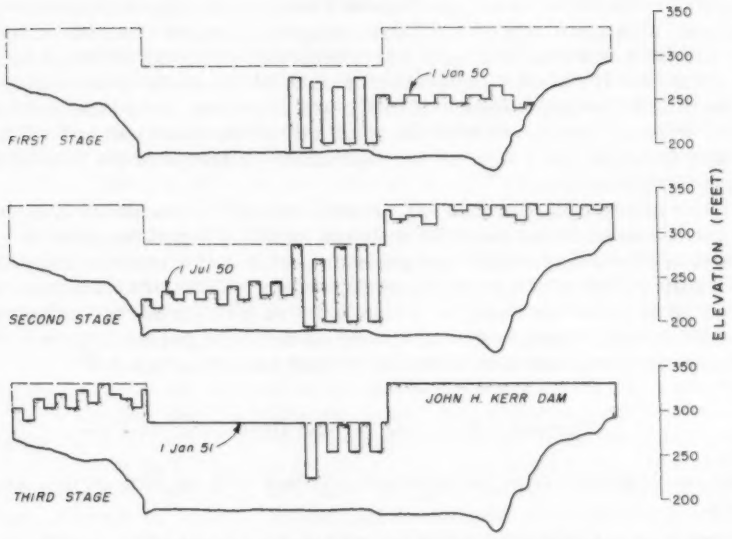


FIG. 1 - TYPICAL CONSTRUCTION PROGRESS WITH RIVER DIVERSION (3 STAGES)

diverted from its normal course by use of a diversion tunnel and the entire area of the dam thus made available for prosecution of the work (Fig. 2).

Other construction procedures such as thickness of lift and the time interval between casting of successive lifts (exposure time) influence the amount of heat dissipated which in turn determines the magnitude of the temperature changes. The thickness of lifts can be rigidly controlled and the exposure time between lifts can be controlled within limits subject to the practical limitations imposed by such things as stream diversion and construction plant capacity. However, when concrete is being placed at initial temperatures of 50 F or less during periods of high ambient temperature, lift thickness has only a minor influence on maximum concrete temperatures, and lift thickness limits may be dictated by construction plant capacities and capabilities, and the problem of placing at the sloping downstream face of a dam.

Temperature changes can be influenced by the use of artificial devices. One of the first such temperature control means used was artificial (or forced) cooling wherein a coolant is circulated through pipes embedded in the concrete. This method has been prominently utilized by the Bureau of Reclamation and to a less extent by the Tennessee Valley Authority, but only to a limited extent by the Corps of Engineers. In its earliest application the method was used primarily for the purpose of cooling a dam to its final stable temperature so that joints could be grouted. Later the idea was extended, notably by TVA at Fontana Dam[5] to include as a primary objective the control of temperature rise.

A second control device extensively utilized is refrigeration, either in conjunction with forced cooling as previously described, or in cooling one or more of the ingredients of the concrete mix so as to lower the placing temperature, thus reducing the maximum temperature and the drop in temperature from maximum to final conditions. In a number of instances

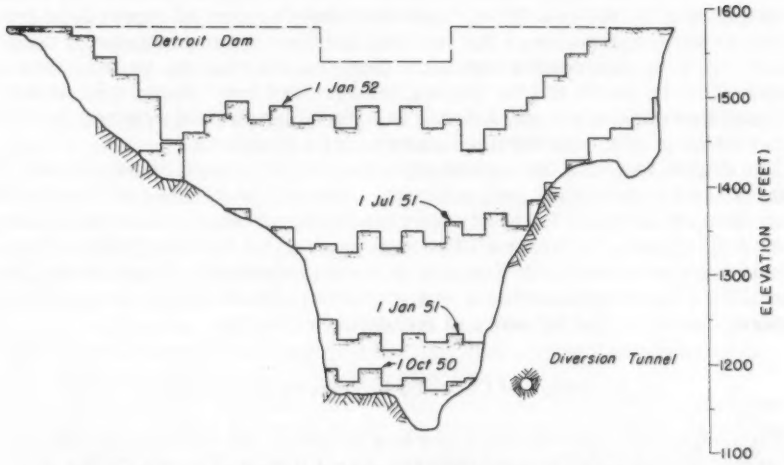


FIG. 2 - TYPICAL CONSTRUCTION PROGRESS WITH TUNNEL DIVERSION

where refrigeration has been a primary control measure it has been supplemented by forced cooling in critical zones within the structure.

Another artificial device which can influence temperature changes is insulation. It has been used in the protection of freshly placed concrete against freezing for many years but only within relatively recent years has insulation been utilized extensively as a temperature control device in prevention of cracking. Its primary application is in protection of surfaces which would otherwise be subjected to sharp and severe drops in temperature.

Thermal properties of the concrete also influence both the magnitude and rate of heat loss and temperature changes, however, the thermal properties of the concrete for all intents and purposes are not subject to control. They are largely determined by the thermal properties of the aggregates and the aggregates used in a structure are largely fixed by availability of suitable materials and economic considerations. In any event the range in thermal properties encountered in practice do not materially influence the control of temperatures except in extreme cases.

Control of Temperature Cracking—Basic Objectives

Simply stated, the basic objectives in control of temperatures to eliminate or minimize cracking in Corps of Engineer dams may be stated as follows:

a. Keep the general cycle of temperature change in the structure as small as practicable.
b. Keep the differences in temperature which prevail at any given time throughout the structure as small as practicable.

In the light of the discussion of restraint and the circumstances which cause cracks to develop it can be seen that the first objective is of primary importance in the base of a dam where foundation restraint is effective. Assuming that foundation restraint varies from 100% at the foundation to as much as 50% at a height above the foundation equal to 15%[5] of the base width then

about 30% of the total concrete in a gravity dam of typical section not divided by longitudinal joints is subject to potential development of appreciable tensile stress as the temperature of the concrete declines from maximum to final stable. To keep this decline small the temperature rise due to cement hydration must be small and the placing temperature low. Under a given set of circumstances these are not difficult to accomplish in cold weather; in warm or hot weather they both become somewhat of a problem.

The degree to which the second objective can be accomplished depends a great deal on construction programming, climatic conditions and the utilization of artificial treatments to modify the changes which would otherwise occur as a result of climatic influences. The second objective is more difficult to plan for and accomplish than the first and in recent experience it is believed that cracks have occurred more as a result of failure to effectively cope with this problem than from the influence of foundation restraint.

Evolution of Corps of Engineers Practices

The control procedures which are now in use by the Corps of Engineers are the result of evolutionary developments over a period of years (Table III). During the period 1933-1940 the Corps constructed among other works five relatively large mass concrete dams. They ranged in height from 150 feet for John Martin to 235 feet for Conchas Dam; and in volume of concrete involved from 337,000 cubic yards for Mahoning Creek Dam to 1,110,000 cubic yards for Tygart Dam. In general the temperature control procedures consisted only of use of moderate heat portland cement (about the equivalent of present day Type II), limiting lift thicknesses to 5 feet with a required minimum time between casting of successive lifts of either 3 days or 5 days and the use of the lowest cement factor then practicable, about 3.5 bags (329 pounds) per cubic yard, for interior concrete.

These practices were not materially different from those followed by others in the USA during this same period for dams of comparable size. Also the experience with respect to frequency of occurrence, size and extent of cracking was about parallel to that of others. In essence frequent, extensive and sometimes rather large cracks occurred on bulkhead faces and tops of lifts exposed for extended periods during fall and winter months. Also both generally vertical and generally horizontal cracking on upstream and downstream faces was frequent and extensive. The extent to which foundation restraint contributed to the start or extension of cracks on bulkhead faces in these dams is not determinable but it is estimated that the average temperature drop in the concrete in the first few lifts above foundation rock placed in summer in these structures may have ranged from 50 F to 60 F which must be characterized as unfavorable[2] even though the decline took place over periods of several years. The experiences during this period indicated that the control procedures utilized were not very effective in eliminating or minimizing cracking.

In the period 1941 to 1945 (World War II) only work of urgent priority on projects contributing to the defense effort was undertaken. Following World War II work on design and construction of dams was immediately accelerated and since that time the Corps of Engineers has completed in addition to other navigation and flood protection works, nineteen major mass concrete dams ranging in height from about 145 feet for John H. Kerr Dam to 440 feet and

460 feet respectively for Pine Flat and Detroit dams. A total of nearly 18 million cubic yards of concrete was placed in the 19 dams ranging from about 300,000 cubic yards in Narrows Dam to 2,150,000 cubic yards in Pine Flat Dam. The volume of concrete in each of 7 of the 19 days is more than 1,000,000 cubic yards.

In the light of previous experience and with the program described above in prospect it naturally followed that the problems of control of temperatures to eliminate or at least minimize cracking was given major consideration in planning for these dams. Soon after the resumption of work in the immediate post World War II period the use of entrained air in all Corps of Engineers concrete was instituted. As an immediate result it was possible to effect substantial reductions in the amount of cement used with a consequent major benefit to control of temperatures. The full potential of entrained air in reducing the amount of cement required however was not realized initially in a number of instances because aggregates, particularly fine aggregates, were not as well or uniformly graded as they might have been.

The next important step therefore in improving on methods of temperature control procedures was to institute the use of well and uniformly graded aggregates with particular emphasis on fine aggregates. This was essential to uniform and sustained use of the lowest practical cement factors in interior concrete where from experience it had been amply demonstrated the lean mass concrete mixes are extremely sensitive to variations in amount and grading of fine aggregate, and is also important in uniform control of air content which is an absolute necessity if required workability is to be maintained in very lean mass concrete.

The combined benefits from use of entrained air and fine aggregate of proper and uniform grading have made it possible to reduce cement factors for interior concrete by more than 25%. Prior to use of entrained air and fine aggregate having optimum grading characteristics for lean mass concrete, cement factors for concrete made with manufactured coarse and fine aggregate averaged about 3.5 bags per cubic yard, and for concrete made with natural coarse and fine aggregate cement factors generally averaged about 3.25 bags per cubic yard. Currently the cement factor for interior concrete made with all manufactured aggregate is 2.5 bags per yard or slightly less; and for all natural aggregate the cement factor is usually 2.25 bags per cubic yard. To achieve a reduction of more than 25% in cement content was a major advancement in the control of temperatures in mass concrete dams. In terms of temperature rise for concrete made with average Type II portland cement and having average thermal properties a reduction in cement from 3.5 to 2.5 bags per cubic yard means a reduction in adiabatic temperature rise of at least 12 F at 28 days. Typical adiabatic temperature rise curves for several projects are shown on Fig. 3.

In seeking further means of controlling temperatures in mass concrete dams the idea of precooling the ingredients to lower the placing temperature was adopted. The first known uses of this idea were by the TVA at Hiwassee Dam[6] in 1938, and by Bureau of Reclamation at Friant Dam[7] in 1940. At Hiwassee the maximum placing temperature in the summer months was reduced from about 78 F to about 72 F by use of refrigerated mixing water. At Friant Dam refrigerated mixing water containing slush ice reduced the placing temperature of the concrete to 70 F in summer. In neither of these cases was there enough reduction in placing temperature to have been of major benefit as a temperature control measure.

TABLE III. CONCRETE TEMPERATURE CONTROL PRACTICES FOR CORPS OF ENGINEERS MASS CONCRETE

STRUCTURE	YEAR STARTED	CONCRETE VOLUME C. Y.	CEMENT PORTLAND	CEMENT REPLACEMENT MATERIAL	FA (Sks
Bonneville	1933	1,000,000	Port.-Pozzolan	-	
Tygart	1935	1,110,000	Mod.to Low Heat	-	3.5
Conchas	1935	830,000	Moderate Heat	-	3.0
Mahoning Creek	1939	337,000	Moderate Heat	-	
John Martin	1940	560,000	Moderate Heat	-	
Norfork	1941	1,500,000	Mod. & Low Heat	-	
Wolf Creek	1941	1,382,000	Moderate Heat	30% Natural	
Bluestone	1942	940,000	Type II	-	
Dale Hollow	1942	574,000	Type II	-	
Center Hill	1942	950,000	Type II	-	
Allatoona	1946	517,000	Type II	20% Slag Cem.	
Fort Gibson	1946	497,000	Type II	-	
Jim Woodruff	1947	425,000	Type II	-	
Narrows	1947	301,000	Type II	-	2.7
John H. Kerr	1948	625,000	Type II	25% Natural	
Clark Hill	1948	1,073,000	Type II	25% Natural	
Mount Morris	1948	741,000	Type II	-	
Harlan County	1948	430,000	Type II	-	
Bull Shoals	1948	2,100,000	Type II	25% Natural	2.5
McNary	1948	1,860,000	Type II(L.A.)	-	
Detroit	1948	1,400,000	Types II & IV	-	
Philpott	1949	326,000	Type II	25% Natural	
Conemaugh	1949	335,000	Type II	-	
Whitney	1949	517,000	Type II	-	
Lookout Point	1949	850,000	Type II	-	
Pine Flat	1950	2,150,000	Type II	-	2.0
Chief Joseph	1951	945,000	Type II(L.A.)	-	
Folsom	1952	1,278,000	Type II	-	
Table Rock	1955	1,170,000	Type II	25% Natural	2.
Ice Harbor	1957	1,250,000	Type II(L.A.)	-	
Sutton	1957	532,000	Type II	30% Fly Ash	2.

FACTOR (Sks /Cu Yd)	SPECIFIED MAXIMUM CONCRETE PLACING TEMP. (DEG. F)	LIFT THICKNESS (FEET)	SPECIFIED EXPOSURE INTERVAL (DAYS) MINIMUM	MAXIMUM	FORCED COOLING	THERMAL INSULATION
3.5	-	5	3	-	-	-
3.5 & 4.5	-	5, 8 & 10	5	-	-	-
3.0 & 3.6	-	5	3	-	-	-
3.4	-	5	5	-	-	-
3	75	5	5	15	-	-
3	-	5	5	-	Lower 10 Lifts-in 32 blocks	
3	75	5	5	-	-	-
3	75	5	5	-	-	-
3.5	-	5	3	-	-	-
3	-	5	3	-	-	-
3	70	5	5	-	-	-
3.5	60	5	5	-	-	-
2.9	65	5	5	-	-	-
2.75 & 3.25	60	5	5	-	-	-
2.5	62	5	5	-	-	-
3	70	5	5	-	Some Lifts on found.rock	-
2.8	70	5	5	-	-	-
3	65	5	5	-	-	6-in. sand
2.5 & 3.0	65	5	5	-	Lower 50 ft. of most blocks	-
2.4	65	5	5	-	-	-
2.5	50	5	5	-	Lower 75 ft. of 8 highest blocks	-
2.5	60	5	5	-	-	-
3	-	5	5	-	-	-
3	75	5	5	15	-	-
2.4	50	5	4	-	-	-
2.0 & 2.25	50	5	5	-	-	-
2.25	50	5	3	-	-	1-in. celotex 2-in. sand
2.5	50	5	5	9	-	-
2.25 & 2.5	50	7 - 1/2	-	12	-	1-in. celotex 2-in. sand
2.25	-	5	5	-	-	-
2.5 & 3.0	50	7 - 1/2	-	12	-	1-in. celotex 2-in. sand

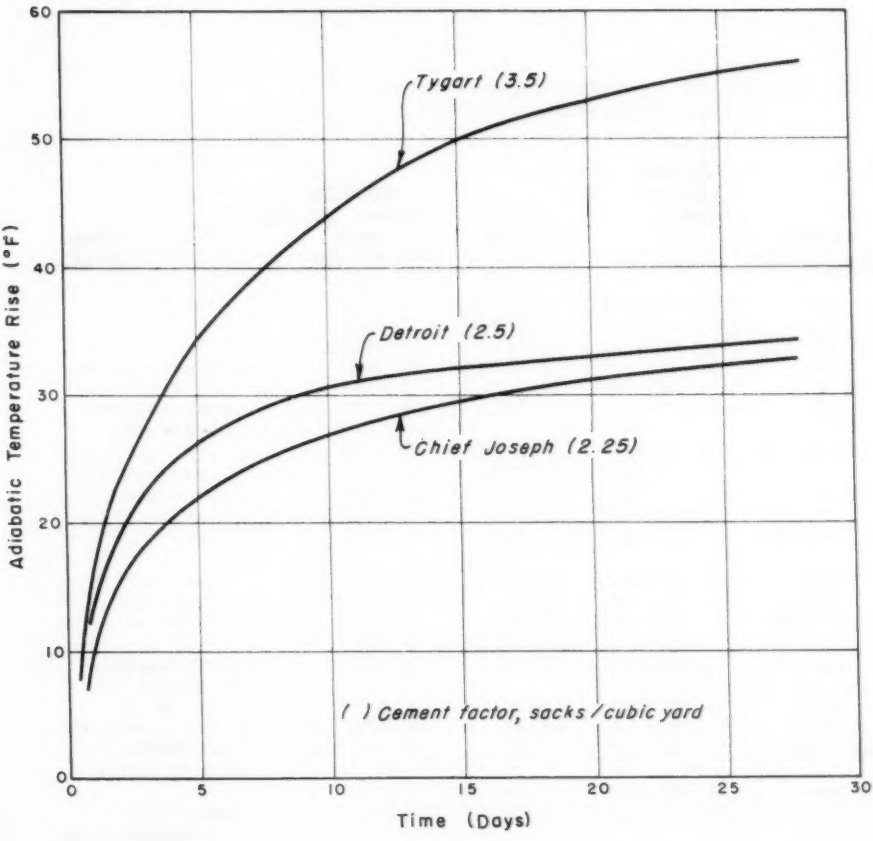


FIGURE 3 - ADIABATIC TEMPERATURE RISE OF MASS CONCRETE

In the development stages of use of precooling by the Corps of Engineers there were uncertainties as to cost and there was no experience in cooling the ingredients in the mix except water. Consequently the limits initially established in specifications as the maximum allowable placing temperature were influenced materially by what was considered the practical limits to which precooling could be accomplished. Typical of the requirements initially was the maximum allowable placing temperature of 65 F set for Bull Shoals Dam started in 1948 and the 62 F maximum set for John H. Kerr (Buggs Island) which was started in 1949 (Table 1). It soon became evident that precooling to temperatures well below those initially specified would be entirely practical by using refrigerated mixing water and ice; and by cooling the aggregates. Economical methods for cooling aggregates were soon developed and

TABLE I - TEMPERATURE CONTROL REQUIREMENTS AND RESULTS

Project	Specified Lift Thickness On Rock and Old Concrete (Feet)	Other Locations (Feet)	Lift Exposure Minimum Specified (days)	Typical Actual (days)	Cement Factor Interior Concrete(a) (bags/cu yd)	Cement Type	Concrete Placing Temperature Range Specified (deg F)	Actual (b) (deg F)	Max Temp of Concrete Placed During Summer	Winter
Clark Hill	2-1/2	5	5	7	3.0	Blend (c)	40-70	45-65	100 (d)	75
Bull Shoals	2-1/2	5	5 (e)	7	2.5	Blend	40-65	42-61	92 (j)	-
Harlan County	2-1/2	5	5	8	3.0	II (f)	40-65	47-65	100	(g)
John H. Kerr	2-1/2	5	5	7	2.5	Blend	40-62	44-60	97	75
Chief Joseph	2-1/2	5	3	5	2.25	II	40-50	42-52	90 (d)	68
Detroit	2-1/2(h)	5	5	6	2.5	II&IV	40-50	41-56	88	72
Pine Flat	5	5	5	6-1/2	2.25	II	40-50	42-52	97	78
Table Rock	7-1/2	7-1/2	(i)	8	2.5	Blend	40-50	42-50	90	71

(a) Cement factor for exterior concrete, 4.0 bags per cu. yd.
(b) Range for more than 90% of concrete placed in the structure.
(c) Portland Type II and Natural Cements in approximately 3:1 proportions.
(d) Estimated.
(e) Reduced to 3 days for 2-1/2 foot lifts placed at 40 F initial temperature.
(f) ASTM designation for Portland Cement Type (II-moderate heat) (IV-low heat).
(g) Placement of mass concrete suspended during winter months.
(h) Five foot lifts where forced cooling was utilized.
(i) Minimum not specified. Maximum limited to 12 days unless insulated.
(j) With forced cooling. Estimated maximum without forced cooling 100 F.

it became feasible for subsequent dams to reduce placing temperatures to 50 F and below during periods of high ambient temperature in the summer months.

The ability to regulate temperature changes in a mass concrete dam by controlling placing temperatures in the range of 45 to 50 F the year round ranks as a major improvement equal in importance to the benefits derived from reducing cement factors by the combined use of entrained air and properly graded fine aggregate. Whereas in summer, placing temperatures formerly ranged from 75 to 85 F and maximum interior temperatures reached 110 to 125 F and higher, with placing temperatures reduced to about 50 F, maximum temperatures rarely exceed 95 F. Mean annual temperatures usually are in the range of 55 to 60 F which means that average drop in temperature from maximum to final stable is reduced from the range 50 to 70 F, to the range 35 to 40 F. The combined influence of the use of entrained air, properly graded sand and precooling is best demonstrated by a comparison of experiences on some earlier projects with the experience on some recent projects (Table II).

The most recently adopted temperature control measure is the use of insulation to protect surfaces which for reasons of construction expediency would otherwise be exposed to very low ambient temperatures during winter months. High bulkhead faces on monoliths adjacent to monoliths left low for stream diversion are one of the prominent situations where the use of insulation is essential if the occurrence of cracks is to be avoided. The tops of lifts in monoliths where regular casting is interrupted for construction reasons during winter months is another situation where insulation is utilized. The influence of insulation on temperature is illustrated in Figs. 4 and 5.

TABLE II - TYPICAL MASS CONCRETE TEMPERATURES

Project	Date Constructed	Constructed By	Temperature at Placement (Summer Conditions) (deg F)	Maximum Interior Temperature (deg F)	Mean Annual Air Temperature (deg F)	Estimated Temperature Drop from Maximum to Final Stable Condition (deg F)
Owyhee (b)	1928-1932	U. S. Bur. Rec.	78 (a)	125	52	73
Boulder (b)	1933-1935	U. S. Bur. Rec.	85	121	72	49
Norris (c)	1934-1936	TVA	88	122	58	64
Hiwassee (d)	1938-1940	TVA	78	112	58	54
Fontana (e)	1943-1944	TVA	77	108	57	51
Detroit	1950-1953	Corps of Engineers	50	88	49	39
Pine Flat	1950-1954	Corps of Engineers	50	97	62	35
Table Rock	1955-1957	Corps of Engineers	49	90	57	33

(a) Data not available. Estimate from ambient air temperature in that locality.
(b) Information from reference 3 in the list at the end of this paper.
(c) Reference 4.
(d) Reference 9.
(e) Reference 10.

Present Practice Summarized

The present practice of the Corps of Engineers for control of temperature to eliminate or minimize cracking in mass concrete dams may be summarized as follows:

a. Cement Type: Type II portland; or a blend of type II portland and either natural or slag cement; or Type II portland and a suitable pozzolan (currently only fly ash); or type ISMH portland blast-furcace slag cement is utilized. The applicable limits on heat of hydration of type II portland cement and type ISMH portland blast-furnace slag cement apply. Influence of use of cement replacement materials on adiabatic temperature rise is shown on Fig. 7.

b. Cement Content: Lowest practicable cement factors are utilized. Cement factors of 2.25 to 2.50 bags per cubic yard are usual for interior concrete; higher cement factors (about 4.0 bags per cubic yard) are used in the outer 5 to 8 feet at the upstream and downstream faces.

c. Placing temperature: Maximum allowable placing temperature is usually fixed at 50 F. (55 F is specified for Hartwell).

d. Insulation: Surfaces to be exposed for long periods (usually 15 days or longer) during winter are required to be protected by insulation.

e. Construction Procedures: Lift thicknesses of 5 or 7.5 feet are utilized. A maximum allowable exposure time (usually 12 to 15 days) is set rather than a minimum exposure. This is designed to insure that casting of successive layers (or lifts) in a monolith will proceed at a regular and uninterrupted rate. Difference in height between adjacent monoliths is limited to three lifts (15 or 22.5 feet as the case may be) except where greater differences are necessary as a construction expedient.

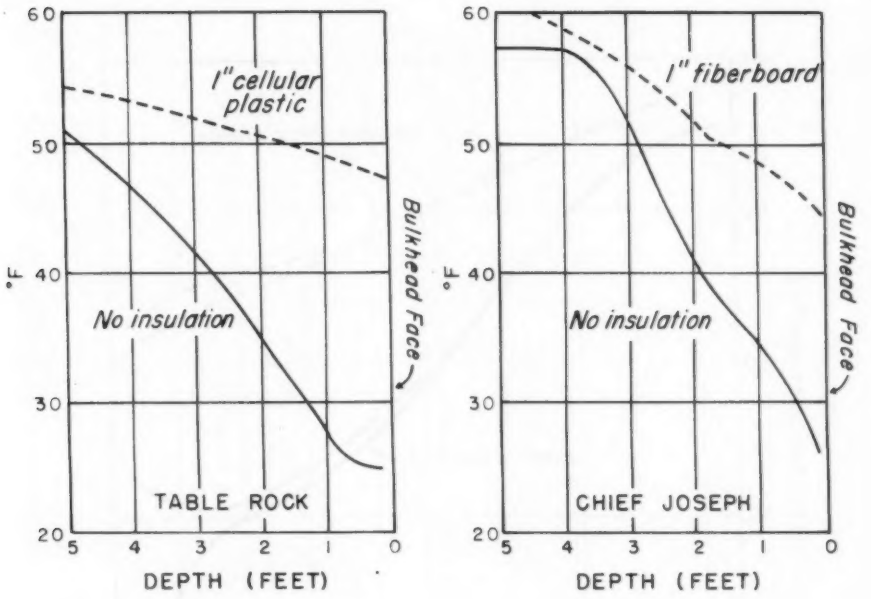


FIG. 4 EFFECT OF INSULATION ON CONCRETE TEMPERATURES NEAR BULKHEAD FACE DURING WINTER EXPOSURE

f. Related Factors: Three important factors contribute materially to the overall control program by making it possible to utilize the extremely low cement factors in the interior concrete. They are, (a) properly and uniformly graded aggregate, particularly fine aggregate; (b) entrained air; and (c) use of heavy duty internal vibrators to compact or consolidate the concrete.

Comparison with Other Procedures

Fundamentally all approaches to the control of temperatures include practices which are more or less basic to the problem. These include the use of lowest practical cement factors, the use of cementing materials of favorable heat generating characteristics and regulation of construction procedures. In the use of cooling, however, there is a matter of fundamental difference between the practice of the Corps of Engineers and that of some other agencies. In the Corps' practice the use of cooling is largely limited to lowering the original (or placing) temperature of the concrete as a means of significantly reducing the maximum temperature experienced in a structure. Other agencies, notably the Bureau of Reclamation and to a limited extent the TVA, have

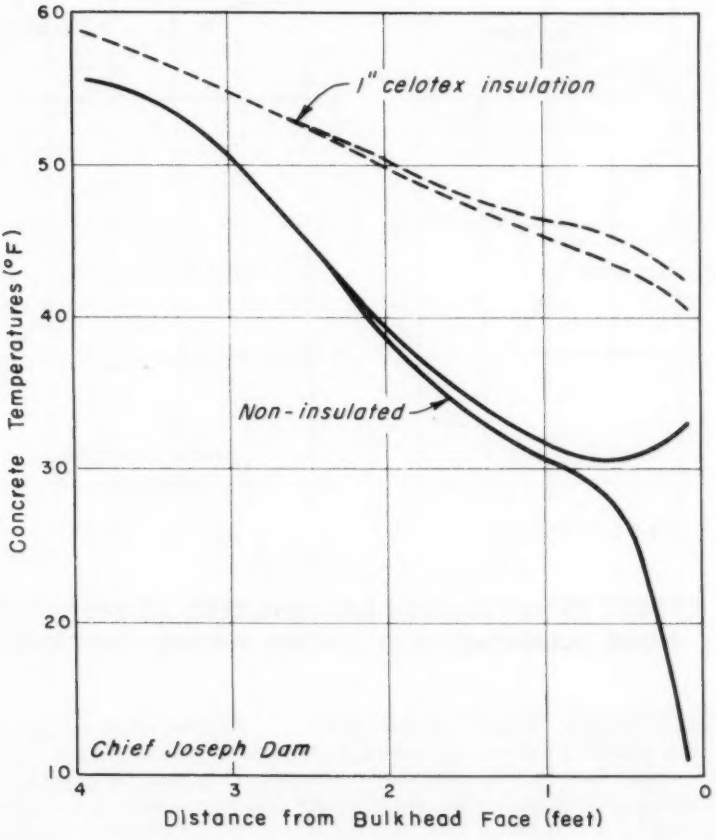


FIGURE 5 - RANGE OF TEMPERATURE GRADIENTS NEAR EXPOSED FACE DURING 24-HOUR PERIOD (AMBIENT RANGE + 15° TO - 8°F)

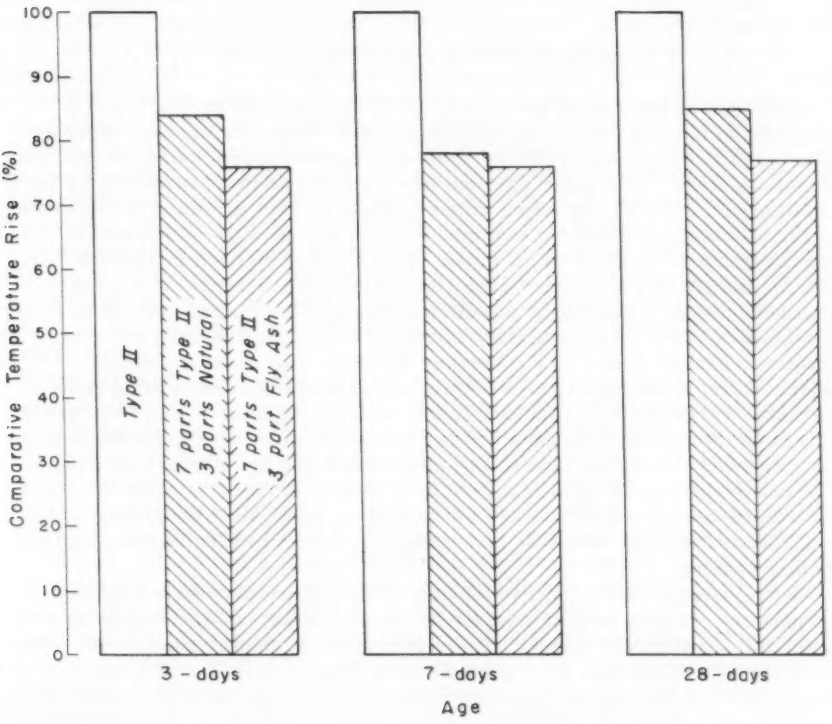


FIGURE 7 - EFFECT OF CEMENT REPLACEMENT MATERIALS ON ADIABATIC TEMPERATURE RISE OF 2¼-BAG MASS CONCRETE

utilized cooling to remove heat from the structure by forced or artificial cooling of the concrete after it is placed in the dam. Because of this fundamental difference in utilization of cooling it is of interest to assess the advantages and disadvantages of the two methods.

The principal advantages of precooling are ease and positiveness of field application and control and low cost. The greatest disadvantage of precooling is that regulation of the temperature cycle is limited to control over the maximum temperature attained.

The advantages of forced cooling (at least theoretically) are its flexibility of application and its control over the entire temperature cycle. The latter when fully effective would obviously be of particular value in regulating interior temperatures so that the large differences in temperature between interior and exposed portions of a dam are avoided. The principal

disadvantages of forced cooling are its complicated and difficult application and control under field conditions and its high cost.

Methods Used in Precooling Concrete

Contractors have developed a number of schemes for precooling the concrete to meet the 50 F placing temperature requirement under summer placing conditions. All of the schemes developed utilize ice and/or refrigerated mixing water. The most efficient pre-cooling method, theoretically, is the addition of all the mixing water in the form of ice; but practical considerations in the efficiency of mixing operations require that at least one-fourth of the mixing water be in the form of water. By using ice and refrigerated water placing temperatures of about 10 F below ambient air temperature can be realized. This, of course, is insufficient to meet the placing temperature requirement of 50 F under summer conditions when average daily air temperatures are usually above 80 F. Further depressing of the concrete mix temperature is obtained by pre-cooling the aggregates. The immersion method[11] was the first method used to cool aggregates. This procedure involves holding each of the three larger sizes of coarse aggregate for periods ranging from 5 to 20 minutes in large tanks containing refrigerated water. The chilled aggregate is then passed over shaker screens to remove some of the excess water, and thence to insulated batch bins. It is not practicable to cool the sand and the smallest size group (#4 to 3/4") of the coarse aggregate by this method.

Next a method of air cooling[12] was developed and has been used rather extensively. In this method cold air is circulated through the coarse aggregates usually after the aggregates are placed in the mixing plant bins. A variation of this method was used at Table Rock Dam, wherein chilled air was passed upward through the separate sizes of the coarse aggregate held in individual 32-foot diameter cylindrical tanks, before delivery to insulated bins in the mixing plant. The air cooling method avoids the necessity for the undesirable water removal operation associated with the immersion method. All four coarse aggregate size groups are cooled; no attempt is made to chill the sand by this method.

On a current Corps project, (Hartwell Dam), a vacuum refrigeration procedure is being used to chill the fine aggregate as well as all four sizes of the coarse aggregate. This method consists of rapidly evacuating a large portion of the air from closed bins containing the moist aggregate, thus reducing the vapor pressure, causing the moisture to vaporize and resulting in a severe decrease in temperature of the solids in the bin. The air (and vapor) evacuation is accomplished by passing a stream jet through a venturi orifice connected to the cooling tanks. The amount of heat withdrawn can be controlled by the velocity of the steam jet, and is not restricted by the heat removal limitations of the cooling medium as are the air blast and immersion methods. Mixing water is chilled by this same procedure.

Experience on Recent Projects

The 19 dams constructed by the Corps of Engineers since World War II are widely dispersed geographically and climatically throughout the United States. Eight of these from which measured temperatures are available represent

reasonably well the group with regard to control procedures and experience. Table I summarizes the specified control procedures and the actual experience on these jobs in achieving the controls intended. The influence of the use of low cement factors (interior concrete) and the use of low placing temperatures is reflected in the relatively low maximum temperatures which occurred in these structures. Also it will be noted that as a result of early experience with precooling it became evident that it would be practical to specify and achieve very low placing temperatures (50 F and lower) in the warmest summer months. Thus whereas in the four projects started first the maximum allowable placing temperature ranged from 62 F to 70 F, the maximum allowable placing temperature on the other four was set at 50 F.

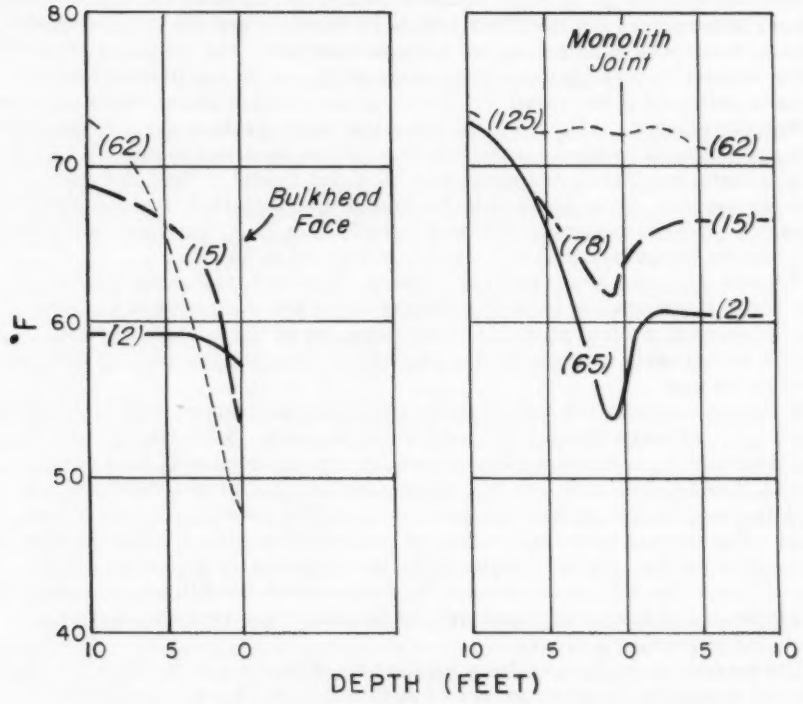
Typical maximum temperatures in three recently constructed dams along with similar information from five earlier dams are summarized in Table II. The influence of control procedures involving use of low cement factors, cement of favorable heat generating characteristics, and low placing temperatures is obvious.

Bulkhead surfaces and tops of lifts exposed during construction to extremely low ambient temperatures are very likely to crack. Such cracks may extend themselves rather than close as cooling and application of load occur. Figs. 4, 5 and 6 illustrate how the temperatures at and near a bulkhead face are influenced by low ambient temperatures and the beneficial effect of insulation. The surface temperature for the insulated condition is considerably higher than for the exposed condition and the temperature gradient is much less severe. The difference between the temperature conditions with and without the benefit of insulation could well be the difference between starting a crack and preventing a crack.

The benefits to be derived from limiting the difference in height between adjacent monoliths so that exposure of bulkhead faces is kept to a minimum, particularly during periods when low ambient temperature may occur, are self-evident. Fig. 6 illustrates temperature conditions which develop while a bulkhead face is exposed and after it is covered by the adjacent monolith. In this case the adverse temperature differentials which existed adjacent to the bulkhead face after the 62-day exposure (admittedly an abnormally long period) were soon modified by the effect of the covering concrete.

The test of a procedure for control of temperature for the purpose of preventing or at least minimizing cracking is the amount, frequency and severity of cracking which occurs. Experience in constructing 19 dams using the control measures described has demonstrated that largely these measures are effective in minimizing cracking and preventing the occurrence of structurally dangerous cracking. In some instances almost total elimination of cracking has been achieved; in at least one case the number of cracks which occurred indicated the procedures were not as effectively applied as would have been desirable. In the latter case some of the cracking which occurred on bulkhead faces would have been prevented if these faces had been effectively insulated. In no case however is there any evidence that the cracking which did occur is structurally significant.

In summary it may be said that cracking in some of the eight representative structures has been almost completely eliminated. Most of the cracks which have occurred are best described as hairline, of very limited extent and of essentially superficial depth. On one of these dams (John H. Kerr, formerly called Buggs Island) not a single crack of even hairline width was observed on any bulkhead. No cracks of any kind have occurred in any of the



Curve numbers in ( ) are ages of concrete in days. Monolith on left placed 28 Sep 51, Monolith on right of joint placed 30 Nov 51.

FIG. 6 TEMPERATURE GRADIENTS NEAR MONOLITH JOINT

galleries of this dam. The absence of cracking in this dam is phenomenal, considering the diversion scheme used (Fig. 1).

Success in eliminating cracking in the other dams listed has not been quite equal to the experience cited for John H. Kerr. However, on the two highest dams constructed since 1945 (Pine Flat and Detroit) the experience was good. At Detroit no vertical cracks were observed on any exposed bulkhead. In this connection it is worth noting (Fig. 2) that with one minor exception all monoliths in the dam were brought up uniformly and progress was regulated to keep the differences in height between adjacent monoliths at a minimum. Consequently, the duration of exposure of bulkhead faces was a practical minimum. In the one exception a single monolith on the left abutment was constructed to a height of about 40 feet as a construction expedient. The bulkhead faces of this monolith were insulated and stood through one winter season without cracking. The only cracking evident at Detroit is two minor horizontal

cracks on the downstream face; and some very minor cracks in the adits and galleries. No cracks of structural significance occurred at Detroit Dam.

Experience at Pine Flat in minimizing cracking was not quite as good as at Detroit though excellent in comparison to earlier experience as typified by the Norris Dam[8] experience. At Pine Flat Dam some vertical cracks, mostly hairline to 0.01 inches in width, were observed on several bulkhead faces following an unusually (for this region) cold period, but there has been no significant cracking noted on the upstream and downstream faces of the structure. The only current evidences of cracking are some leakage in the intake monoliths and a few minor cracks in galleries and adits. Here again, considering the magnitude of the structure, the relatively small number, size and extent of the cracks which occurred is gratifying evidence of the effectiveness of the controls utilized. It should be noted that the use of insulation was not a general practice when Pine Flat was constructed and the cracking on bulkheads described probably would have been avoided if these had been protected by insulation. No cracking of structural significance has occurred at Pine Flat.

Not all of the experience in control of cracks has been as successful as at John H. Kerr, Pine Flat and Detroit Dams. One exception is at Chief Joseph Dam, where vertical cracking occurred on eleven bulkhead faces and horizontal and vertical cracks have developed on the upstream and downstream faces in 20 of the 27 monoliths. Even though occurrence of cracking was more frequent and the cracks were more extensive than in other dams built by the Corps of Engineers during the period, no cracking has occurred which is structurally dangerous.

CONCLUSIONS

On the basis of these experiences the following conclusions seem justified:

a. It is both possible and practical to prevent structurally significant temperature cracking in mass concrete dams.
b. The temperature control practices utilized by the Corps of Engineers have been notably effective in preventing structurally significant cracking.

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FIELD STUDY OF INTERIOR TEMPERATURES IN CONCRETE

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ABSTRACT

Interior temperatures attained by concrete in a thin arch and gravity dam are reported. Cement contents ranged from 2 to 4.75 sk/cu yd. Compressive strengths ranged from 1600 to 4285 psi at 28 days. The mean "Coefficient of maximum temperature increase" for the concrete was 11° F per sk of design cement content.

SYNOPSIS

Data obtained on interior temperatures attained by mass concrete in two types of concrete dams is reported, one a thin arch type structure, the other a massive gravity type structure. Cement contents of the concrete investigated ranged from 2.0 to 4.75 sk/cu yd, and average compressive strengths ranged from 1610 to 4285 psi at 28 days. The "coefficient of maximum temperature increase" based on the design cement content of the concrete investigated ranged from 6.1 to 15.3° F/sk of cement content, the mean coefficient being about 11° F/sk.

INTRODUCTION

The buildup of heat in the interior of massive concrete sections due to heat of hydration of the cement has been the subject of extensive field and laboratory investigation since the construction of Hoover Dam in the late 1920's and the early 1930's. The subsequent cooling of concrete structures having elevated temperatures results in volume changes and subsequent contraction

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cracking if restraint to volume change exists and if stresses induced by these volume changes exceed the tensile strength of the concrete. The property of plastic flow which is a physical characteristic that permits concrete to yield upon application of external stress with absence of the development of an equal resisting stress within the concrete eliminates much of the cracking tendency resulting from heating and cooling volume change. Some investigators[1] consider that the effect of plastic flow increases from zero at the rock foundation and old concrete foundation (concrete more than one month old) to unity at a distance of 0.5 H or 0.6 H above rock or old concrete where "H" is the height from the rock or concrete foundation to the top of the structure.

Since the initiation of construction on Hoover Dam two basic methods have been developed and utilized to control heat generation in massive concrete structures, namely: (1) Control of heat generation after the concrete is placed by an embedded cooling system, or (2) Control of the maximum temperature that the concrete will attain in place by control of the initial temperature of the concrete when placed, the use of the lowest practical cement content,[2] the use of special low heat cements and the use of relatively thin lifts of concrete with relatively long intervals of time between successive lifts. In some cases combinations of both methods have been utilized. The "Construction Slot" method for controlling the tightness of construction joints in past years, particularly for arch dams also afforded a means of reducing maximum temperatures attained by the concrete in place. This method consisted of leaving narrow openings through the cross section of the dam at the construction joints which were filled at a later time when the concrete had cooled and the principal contraction had occurred. The exposure of the contraction joint ends of each block of concrete to the outside air aided in the dissipation of heat from the concrete during the construction period. The principal of grouting the construction joints in arch dam structures after cooling of the concrete has occurred is still utilized. However, the practice of forming "Construction Slots" in the dams at the construction joints to be filled with concrete or mortar at a later time has been largely discontinued.

In recent years, the second basic method of control described above has been utilized extensively.[3,4] Under this second method all or some of the following practices are utilized:

(1) The initial placed temperature of the concrete is reduced by utilizing cool ingredients, cool mixing water (sometimes refrigerated), chipped ice as a replacement for some of the mixing water, cool aggregates (cooled by means of sprinkling or refrigeration) and cool cement (cooled sometimes by aeration);
(2) The lowest practical cement content (sometimes in conjunction with a pozzolanic replacement)[5,6,7] is utilized;
(3) The utilization of a Type II cement, a modified low heat and sulphate resistant cement, is universally practiced, and occasionally when the extra cost is considered to be justified, Type IV, a special low heat cement, is utilized;
(4) Thin lift heights (5 ft to 10 ft for usual sections and 2-1/2 to 5 ft for the first two to four lifts off rock or off old concrete (more than one month old) are utilized; and
(5) Intervals of up to 120 hours (5 days) between successive lifts are utilized to reduce maximum temperature rise and reduce the heat potential

of the concrete in place before it is covered by additional insulating lifts of concrete.

The North Fork and Noxon Rapids Dams now being constructed in Northwestern Oregon and Northwestern Montana, respectively, afforded an opportunity to obtain important field heat generation data for two different types of concrete structures. Noxon Rapids represents a massive gravity type structure utilizing lean interior concrete and richer face concrete, whereas, North Fork represents a relatively thin arch type structure utilizing a richer face type concrete throughout the complete structure. The heat generation in the concrete was controlled for both structures by the second method described above, that of utilizing the coolest practical concrete with the use of cool ingredients, the use of a Type II, moderate low heat sulphate resistant cement, the limitation of lift height off foundation or concrete more than one month old, the control of the time between successive concrete placements and the use of relatively low cement contents. At North Fork, flake ice was utilized to reduce the initial concrete temperature during severe hot weather. No ice was utilized at Noxon.

Internal temperatures of the concrete in place were observed by means of Carlson Type MA resistance thermometers and test sets.

Description of Structures

The North Fork Hydro-electric Development is located on the Clackamas River about 30 miles Southeast of Portland, Oregon. The development is comprised of an arch dam 675 ft in length and 207 ft high above bed rock, a spillway structure abuting the arch dam on the right bank of the river having a 3 gate, 150 ft long overflow section, with a total capacity of 150,000 cfs, and a powerhouse having two turbo-generating units having a generating capability of 56,000 Kw. Also included in the development are fish passing facilities abuting and extending upstream from the spillway structure on the right bank and a fish ladder extending approximately 10,000 ft downstream to just beyond the existing Cazadera dam. The principal sections of the arch dam (excluding sections around power penstocks, sluiceways, or at the roadway deck on the top of the dam) on which interior concrete temperatures were measured, range from 32 ft wide at the base to 8 ft wide at the top.

The Noxon Rapids Hydro-electric Development is located in Northwestern Montana on the Clark Fork river at the northern tip of the Bitterroot Range about equidistant between Missoula, Montana and Spokane, Washington. The Development is comprised of a combination concrete and earthfill dam 5854 ft in length, a powerhouse located directly downstream having an ultimate rated capacity of 500,000 Kw in five turbo generator units, 30 miles of State highway, and county road relocation and 18 miles of railroad relocation. The dam consists of a concrete gravity structure in the center 1054 ft in length and 260 ft high above bed rock flanked on each abutment by a total of 4800 lineal ft of earth embankment having a clay core. The concrete dam contains a 502 ft long intake structure 138 ft wide at the base and 18.5 ft wide at the top, excluding the road deck, a 384 ft eight-tainter gate overflow section having a total discharge capacity of 250,000 cfs, and two bulkhead structures, one on each end of the concrete dam, having lengths of 78 and 90 ft. The maximum width of the spillway structure at the base excluding the downstream apron section is 145 ft.

Photographs of the North Fork arch dam and powerhouse and the Noxon Rapids dam and powerhouse are shown in Figs. 4 and 5.

Concrete Mix Design

The design constants and compressive strength yield for the concrete mixtures from which temperature data was obtained are shown in the attached Table 1 and Fig. 1. As shown in these displays the cement contents of the concrete at North Fork on which internal temperature observations were made ranged from 4 to 4.75 sk/cu yd, the water cement ratios ranged from 0.47 to 0.52, and the average compressive strengths for these mixtures ranged from 3430 to 4285 psi at 28 days and from 4920 to 5805 psi at 3 months. For Noxon the cement content of the interior lean concrete on which temperature observations were made ranged from 2 to 2.5 sk/cu yd, and the water cement ratio ranged from 0.65 to 0.85. The face concrete had a cement content of 3.75 sk/cu yd and a water cement ratio of 0.47. The average compressive strength for the lean concrete ranged from 1610 to 2310 psi at 28 days, from 2000 to 2715 psi at 3 months, and from 2130 to 2965 psi at 6 months. Face concrete strength averaged 4145 psi at 28 days, 5285 psi at 3 months, and 5945 psi at 6 months.

The minimum compressive strength specified for face concrete at Noxon Rapids was 3000 psi at 28 days and the maximum specified water cement ratio was 0.50. For mass interior concrete, a minimum compressive strength of 1800 psi at 28 days and a maximum net water cement ratio of 0.75 was specified, and for lean fill concrete no minimum compressive strength was specified but a maximum net water cement ratio of 0.85 was specified. For North Fork the minimum compressive strength specified for the arch dam was 4000 psi at 3 months, and for all other exposed concrete structures, it was 3000 psi at 28 days.

Initial proportions for laboratory design of the concrete mixtures utilized at North Fork and Noxon Rapids were based on ACI Standard 613-54.(8) Final proportions were selected from a series of laboratory trial mixtures having varying aggregate proportions and cement content but constant water cement

TABLE 1 - MIX DESIGN CONSTANTS AND STRENGTHS FOR CONCRETE ON WHICH HEAT GENERATION STUDIES WERE MADE AT THE NORTH FORK AND NOXON RAPIDS DEVELOPMENT

Mix Identification	Mix Design Constants						Average Compressive Strength at Age Indicated, psi			
	Max Aggr Size In	Cement Content sk/cu yd	Net w/c	Sand Content %	Slump Range In	Air Content Range %	7d	28d	3 mo	6 mo
North Fork Arch Dam										
6-4 (1)	6	4.00	0.52	26	1½-2½	3.5± 1	(59) 1640	(60) 3430	(60) 4920	---
6-4.25 (1)	6	4.25	0.49	25	1½-2½	3.5± 1	(174) 2095	(123) 4185	(124) 5495	---
3-4.75 (1)	3	4.75	0.49	30	1½-2½	4± 1	(31) 2035	(30) 4150	(30) 5100	---
6-4.50 (1)	6	4.50	0.47	24	1½-2½	3.5± 1	(14) 2165	(13) 4285	(14) 5805	---
Noxon Rapids Dam										
A6-3.75 (1)	6	3.75	0.47	26	1½-2½	3± 1	(417) 2430	(413) 4145	(4) 5285	(3) 5945
B6-2.5 (1)	6	2.50	0.65	28	1½-2½	3.5± 1	(270) 1115	(194) 2310	(5) 2715	(2) 2930
B6-2.25 (1)	6	2.25	0.72	28	1½-2½	3.5± 1	(142) 1005	(130) 2160	(29) 2465	(3) 2965
X6-2 (1)	6	2.00	0.85	28	1½-2½	3.5±	(30) 805	(30) 1610	(5) 2000	(5) 2130

Explanation of Symbols:
1. Net w/c refers to the ratio of net mixing water to cement by weight in the mix.
2. Sand Content, %, refers to the percentage of sand in the total aggregate in the mix by weight.
3. Air Content, %, is based on the total volume of the mix.
4. Numbers in parentheses above average strength values indicate the total number of test results included in the average.

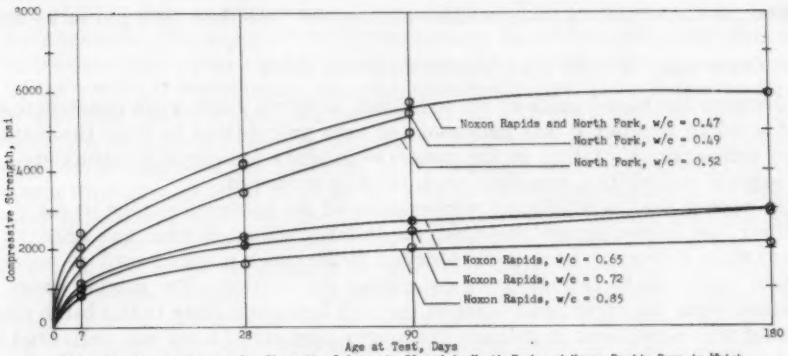

Figure 1 - Average Compressive Strength of Concrete Placed in North Fork and Noxon Rapids Dams in Which Carlson Type MA Resistance Thermometers Were Embedded

ratio, entrained air content and slump ranges. All cement utilized in trial mixtures and field produced concrete was Type II, low alkali.

An evaluation of the compressive strength yield of field produced concrete was made based on ACI Standard 214-57.(9) The results of this evaluation are given in Table 2. In summary this evaluation showed that the average compressive strength for face concrete at Noxon Rapids was controlled such that about one (1) cylinder in 50 would yield a compressive strength of less than 3000 psi at 28 days, whereas, at North Fork, the average compressive strength of the concrete was controlled such that about one (1) cylinder in 15 would yield a compressive strength of less than 3000 psi at 28 days. The difference in compressive strength yield for the exterior concrete utilized on the two projects was due to the lower water cement ratio used at the Noxon Rapids Project where a higher durability requirement for resistance to the effect cycles of freezing and thawing and severe weather was deemed necessary. For interior concrete at Noxon Rapids in which durability was no factor, the

TABLE 2 - SUMMARY OF EVALUATION OF FIELD COMPRESSIVE STRENGTH YIELD OF CONCRETE UTILIZED IN HEAT GENERATION STUDIES AT THE NORTH FORK & NOXON RAPIDS HYDRO ELECTRIC DEVELOPMENTS IN ACCORDANCE WITH ACI STANDARD 214-57

Mix Identification	Grand Av Compr St psi	Standard Deviation (σ) From Average psi	Coefficient of Variation (V) From Average %	No of Test Below Specified Compr Str	Required Average Compressive Strength (frc) Based on ACI 214-57			
					For 1 Failure in 5 Tests	For 1 Failure in 10 Tests	For 1 Failure in 20 Tests	For 1 Failure in 50 Tests
Noxon Rapids Face Concrete, Evaluation Based on Specified Minimum Strength of 3000 psi at 28 Days								
A6-3.75 (1)	4145	450	11.3	9 in 441	Not Applicable	2540	3720	3930
North Fork Face Concrete, Evaluation Based on Specified Minimum Strength of 3000 psi at 28 Days								
A6-4.5 (1)	4285	365	8.5	0 in 13	Not Applicable	3390	3510	3690
A6-4.25 (1)	4185	530	12.7	2 in 123	"	3600	3810	4080
A3-4.75 (1)	4150	530	12.7	2 in 30	"	3600	3810	4080
A6-4 (1)	3430	515	15.2	11 in 60	"	3750	4020	4410
North Fork Face Concrete, Evaluation Based on Specified Minimum Strength of 4000 psi at 3 mo								
A6-4.5 (1)	5805	690	11.9	0 in 14	Not Applicable	4720	5000	5365
A6-4.25 (1)	5495	520	9.6	0 in 124	"	4600	4800	5040
A3-4.75 (1)	5100	465	9.1	1 in 30	"	4560	4760	4960
A6-4 (1)	4920	595	14.2	1 in 60	"	4880	5200	5720
Noxon Rapids Interior Mass Concrete, Specified Minimum Strength of 1800 psi at 28 Days								
B6-2.5 (1)	2310	300	13.1	18 in 286	2025	2125	Not Applicable	Not Applicable
B6-2.25 (1)	2160	440	16.0	22 in 112	2090	2270	"	"
Noxon Rapids Lean Fill Concrete (No Req'd Strength Specified) Evaluation Based on 1800 psi at 28 Days								
X6-2 (1)	1610	233	14.5	25 in 30	2055	2215	Not Applicable	Not Applicable
Noxon Rapids Lean Fill Concrete (No Req'd Strength Specified) Evaluation Based on 1800 psi at 6 mo								
X6-2 (1)	2130	392	18.4	1 in 5	2140	2360	Not Applicable	Not Applicable

average compressive strength was controlled such that about one (1) cylinder in five (5) would yield a compressive strength of less than 1800 psi at 28 days.

Heat Generation in Arch Dam Concrete, North Fork

Because the base blocks of the arch dam at North Fork were constructed during warm weather it was necessary to take precautions to limit the maximum temperature attained by the concrete in place to reduce or eliminate subsequent contraction cracking when cooling occurred.

To reduce the initial placed temperature of the concrete during warm weather, the ingredients of the concrete (mixing water, cement and aggregates) were delivered from their sources to the batch plant as cool as possible without resorting to refrigeration equipment for cooling. The mixing water obtained from the river, was pumped through insulated lines to the batch plant. Cement which had been in storage the longer periods of time was delivered to the project by the cement vendor. Coarse aggregates were sprinkled in the storage piles and rinsed on the re-screen to obtain cooling by evaporation during delivery to the batch plant. In addition during specific periods of severe hot weather flake ice in quantities of from 35 to 70 lb per cubic yard of concrete were added to concrete batches to accomplish an additional reduction in the initial concrete temperature. Moist curing by water spray was utilized throughout the entire construction period, and heat dissipation from the concrete after placement was increased in hot weather by the loosening and removal of forms at the earliest practical time. Concrete mixtures having relatively low cement contents, with due consideration for required strength and durability, were utilized to reduce the total heat generated by the concrete in place. The cement contents, as given previously in this report, varied from 4.00 to 4.75 sk/cu yd.

To aid in the planning for control of internal concrete temperatures, fifteen Type MA Carlson resistance thermometers were embedded in the interior of fifteen separate lifts of concrete placed in the arch dam at North Fork during

TABLE 3A - INTERIOR HEAT GENERATION IN 10 FT LIFTS AS DETERMINED BY CARLSON TYPE MA RESISTANCE THERMOMETERS EMBEDDED IN THE CONCRETE - NORTH FORK HYDRO ELECTRIC DEVELOPMENT

Heat Generation in the Interior of Concrete Having Physical Characteristics as Listed Below, °F									
Time After Placement (Days)	10 Ft Lift, 50 Ft Wide; Sec Less than 20 Ft thick; Placed in Fall; 4 to 4.5 sk/cu yd			10 Ft Lift, 50 Ft Wide; Sec more than 20 Ft Thick; Placed in Fall; 4 sk/ cu yd			10 Ft Lift, 50 Ft Wide; Sec more than 20 Ft Thick; Placed in Summer; 4.25 to 4.5 sk/cu yd		
	Therm - 1A, 2B, 12A			Therm - 3A, 4B, 5B			Therm - 7A, 9A		
	Max	Min	Ave	Max	Min	Ave	Max	Min	Ave
0	56	51	54	63	56	59	72	65	68.5
2	94.1	83.2	87.8	—	—	—	111.9	108.1	110.0
4	99.1	92.1	95.6	95.3	78.3	89.1	123.3	114.6	119.0
6	104.8	97.1	101.6	99.3	90.3	95.9	125.5	119.3	122.4
8	103.9	100.2	102.0	100.8	99.9	100.4	—	—	—
10	102.0	101.5	101.8	100.7	98.5	100.5	122.6	116.5	119.6
15	103.9	96.9	100.4	103.2	100.3	102.2	116.4	110.9	113.7
20	104.6	92.4	99.7	105.8	104.5	105.2	110.2	105.8	108.0
30 (1 mo)	102.3	82.1	94.8	107.6	104.4	106.1	100.7	97.4	99.1
60 (2 mo)	89.5	72.4	82.9	99.2	90.3	95.2	85.2	76.4	80.8
90 (3 mo)	76.4	63.1	70.0	87.4	75.0	81.8	80.1	72.8	76.5
120 (4 mo)	65.9	53.7	59.8	73.2	68.1	70.7	76.0	69.8	72.9
180 (6 mo)	58.2	48.9	53.6	—	—	59.7	65.7	57.0	61.4
270 (9 mo)	54.2	52.9	53.6	60.7	56.1	58.1	—	—	—
320 (11 mo)	—	—	—	—	—	—	57.7	53.0	55.3
Max Temp Rise °F	53.6	45.5	48.0	51.6	41.5	47.1	60.5	47.3	53.9
Mixing Water Temp Range °Fe	48-50			44-50			60-62		
Air Temperature Range °F	(55.5 Mean) 65-46			(55.5 Mean) 65-46			(67 Mean) 84-50		
Coef of Max Temp Incr °F/sk	(11.3 Av) 10.1-13.4			(11.8 Av) 10.4-12.9			(12.3 Av) 10.5-14.2		
% of Total Cone Surf Area Exposed During Observ Period**	(66.0% Mean) 38 to 93%			(68.5% Mean) 37 to 100%			(65.5% Mean) 31 to 100%		

Notes: * 1. During day of placement of concrete.

**2. Total potential exposed surface is summation of areas, (1) upstream and downstream faces; (2) two ends, and (3) top horisontal construction joints. For Comnlete Summary see Table 5.

initial periods of heavy placement. Temperature observations obtained from twelve of these thermometers are summarized in Tables 3A and B and Fig. 2 of the Appendix. The other three thermometers failed to yield useful data due to instrumentation errors and loss of access after installation.

Of the twelve thermometers yielding significant data, eight thermometers were embedded in 10 ft high concrete lifts and four were embedded in 5 ft high lifts. Eight thermometers were embedded in concrete sections having a thickness of more than 20 ft but less than 35 ft, and four were embedded in sections having a thickness of from 15 to 20 ft. Five of the thermometers were embedded in lifts placed in summer months; six were embedded in lifts placed in the fall; and one was embedded in a lift placed in the winter. All of the concrete placed in the sections of the arch dam which were observed for internal temperature change by resistance thermometers yielded a compressive strength in the order of 5000 psi or higher at an age of 3 months. All lifts in which thermometers, were embedded were in the order of 50 ft in length.

TABLE 3B - INTERIOR HEAT GENERATION IN 5 FT CONCRETE LIFTS AS DETERMINED BY CARLSON TYPE MA RESISTANCE THERMOMETERS EMBEDDED IN CONCRETE NORTH FORK HYDRO ELECTRIC DEVELOPMENT

Heat Generation in the Interior of Concrete Having Physical Characteristics as Listed Below, °F				
Time After Placement (Days)	5 Ft Lift, 50 Ft Wide; Sec more than 20 Ft thick; Placed in Summer; 4 to 4.25 sk/cu yd Therm 2A, 4A, 5A			5 Ft Lift, 50 Ft Wide: Sec less than 20 Ft thick; Placed in Winter; 4.75 sk/cu yd Therm - TB-2A
	Max	Min	Ave	
0	68	58	64	52
2	98.7	82.0	90.7	83.2
4	98.2	90.4	93.2	94.9
6	92.5	90.6	91.6	97.0
8	100.0	90.0	95.0	99.6
10	102.5	88.1	97.0	100.9
15	107.1	90.2	99.3	95.1
20	106.1	96.2	101.2	89.8
30 (1 mo)	109.5	97.2	102.4	80.5
60 (2 mo)	--	--	94.2	60.2
90 (3 mo)	--	--	84.6	58.4
120 (4 mo)	81.4	76.3	79.0	--
180 (6 mo)	67.5	64.5	65.8	60.8
270 (9 mo)	--	--	--	--
320 (11 mo)	--	--	58.9	--
Max Temp Rise °F	51.5	29.2	38.2	48.9
Mixing Water Temp Range °F*	50-57			--
Air Temp Range °F	(67 Mean) 84-50			(45.5 Mean) 53-38
Coef of Max Temp Incr °F/sk	(9.3 Av) 6.9-12.9			(10.3 Av) 10.3
% of Total Conc Surf Area Exposed During Observ Period**	(56.5% Mean) 19 to 94%			(66.0% Mean) 41 to 91%

Notes: * 1. During day of placement of concrete.
**2. Total potential exposed surface is summation of areas, (1) upstream and downstream faces; (2) two ends, and (3) top horizontal construction joints. For Complete Summary see Table 5.

All resistance thermometers were embedded in the center of the mass of the lifts such that their distance from the exposed end or ends of the lifts ranged from 22.5 to 25 ft, their distance from the upstream or downstream faces ranged from 8 to 17.5 ft, and their distance from the top of the exposed construction joint was 5 ft for 10 ft lift heights and 2-1/2 ft for 5 ft lift heights. On the basis that internal heat generated in the concrete lifts observed could be dissipated from ends, sides, and the top exposed construction joints, the percentage of the total area of these surfaces not covered and therefore exposed for heat dissipation during the observation period ranged from 19 to 100 per cent. The maximum exposure of surfaces for heat dissipation usually occurred at about 2 days after placement of the concrete when the forms were stripped and the top construction joints were exposed. The duration of this maximum exposure for lifts observed at North Fork ranged from 1 to 42 days, the usual period being less than 5 days. The minimum exposure of surfaces from which heat could be dissipated occurred when additional lifts had been placed over and against those being observed such that only the upstream and downstream faces were exposed. The time at which minimum exposure occurred in the lifts observed at North Fork ranged from 5 to 72 days after placement of concrete in the lift, and the time extended for the remainder of the observation period. A complete tabulation of exposure conditions for the concrete lifts on which internal temperatures were observed is given in Table 5.

The highest recorded internal temperature attained in a specific lift of concrete in the arch dam at North Fork was 125° F. This lift was 10 feet high, 23 feet thick, and it was placed at an initial temperature of 72° F in the summer months. The concrete had a cement content of 4.25 sk/cu yd. The lowest recorded maximum internal temperature attained in a specific lift of concrete was 101° F. This lift was 5 feet high, 18 feet thick, and it was placed at an initial temperature of 52° F in the winter months. The concrete in this lift had a cement content of 4.75 sk/cu yd.

The total temperature rise in the concrete of the arch dam resulting from internal heat generation regardless of size of section, lift height, cement

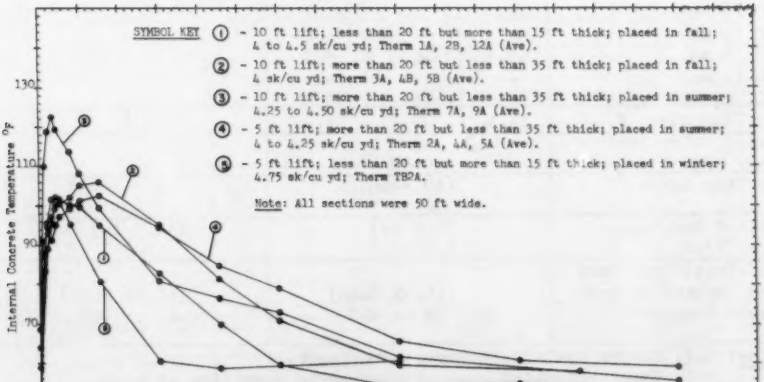
SYMBOL KEY
① - 10 ft lift; less than 20 ft but more than 15 ft thick; placed in fall; 4 to 4.5 sk/cu yd; Therm 1A, 2B, 12A (Ave).
② - 10 ft lift; more than 20 ft but less than 35 ft thick; placed in fall; 4 sk/cu yd; Therm 3A, 4B, 5B (Ave).
③ - 10 ft lift; more than 20 ft but less than 35 ft thick; placed in summer; 4.25 to 4.50 sk/cu yd; Therm 7A, 9A (Ave).
④ - 5 ft lift; more than 20 ft but less than 35 ft thick; placed in summer; 4 to 4.25 sk/cu yd; Therm 2A, 4A, 5A (Ave).
⑤ - 5 ft lift; less than 20 ft but more than 15 ft thick; placed in winter; 4.75 sk/cu yd; Therm TB2A.
Note: All sections were 50 ft wide.
Internal Concrete Temperature °F
130
110
90
70
50
40
80
120
160
200
240
280
320
Time After Placement, Days

Figure 2 - Heat Generation in Interior Concrete as Observed by Means of Carlson Type MA Resistance Thermometers Embedded in Arch Dam, North Fork Hydro-Electric Development

content, weather conditions during placement, or surface area exposure conditions based on data obtained from the embedded resistance thermometers, ranged from 29.2 to 60.5° F. When this range of maximum internal temperature development is analyzed on the basis of the design cement content of the concrete placed, the temperature increase range can be reduced to "Coefficients" ranging from 6.9 to 12.9° F rise/sk of cement.

As can be noted in Tables 3A and B and Fig. 2, no significant difference in the order of total increase of internal temperature was noted between 10 foot high lifts having cross-sections of more than 20 feet thick and those less than 20 feet thick (but more than 15 ft thick). The 10 ft lifts placed in the summer showed a maximum increase in temperature of about 5 to 10° more than comparable 10 ft lifts placed in the fall. The total increase in temperature for all 10 ft lifts observed ranged from 41.5 to 60.5° F, and the "Coefficient of Maximum Temperature Increase" ranged from 10.1 to 14.2° F rise per sk of cement. Sections having 5 ft lift heights had total temperature increases which were in the order of 10° F less, and "Coefficient" in the order of 1.5 to 3° F/sk less than comparable 10 ft lifts.

The temperature generation and dissipation cycle for the arch dam concrete was relatively rapid. For the blocks observed with the twelve test thermometers approximately 80 - 90 per cent of the maximum internal temperature was attained in 48 hr (2 da), and 91 to 99 per cent in 120 hr (5 da). All blocks had attained their maximum temperature by 30 days after placement. A reduction in maximum temperature had begun in most of the blocks in 240 hr (10 da) and in all blocks by 30 days after placement. In addition a stabilized temperature near the mean annual temperature for this region was attained in a period of from 6 to 8 months after placement.

In analyzing the range of mixing water temperatures used in the concrete and the ambient air temperatures at the time of placement of the concrete, it is significant to note that the initial temperature of the concrete placed at North Fork more nearly approached that of the mean air temperature. The exception to this trend was shown for concrete placed in winter weather when an effort to use cool ingredients was not made.

As a field guide for future projects having concrete sections of sizes comparable with those of the arch dam at North Fork (5 to 10 ft lift heights, 15 to 35 ft thick cross sections, and 50 ft lengths) and utilizing Type II cement and no cooling coils, a satisfactory mean coefficient for estimating the order of maximum internal temperature increase to be expected for the concrete in place appears to be about 10.5° F/sk of cement. For example it would be expected that 2 sk/cu yd concrete would increase in temperature in place in the order of 20° F and that 6 sk/cu yd concrete would increase about 65° F.

No cracking attributable to volume change resulting from internal heat generation and dissipation has appeared to date in the sections in which resistance thermometers were embedded at North Fork. One lift in the main section of the arch dam in which no thermometer was embedded has exhibited one vertical and horizontal crack at a time when the internal temperature of the concrete was apparently approaching the mean annual temperature for the region, and some cracking has occurred in curb and roadway sections at the top of the dam. There appears to be no relationship between the cracking in the road deck slabs and curbs and heat generation and dissipation. However, for the one lift of the arch dam cited above which has exhibited cracking, there appears to be reasonable indication that the cracking was due to excessive tensile stresses resulting from excessive heat generation and dissipation. This lift which was 10 ft high and approximately 23 ft thick and 50 ft

long was placed in the summer within two days after another lift having similar size, mix and initial temperature (72° F) characteristics but exhibiting no cracking after attaining a maximum internal temperature measured by a Carlson Thermometer of 125° F. The lift exhibiting cracking, was in the order of 0.20 H above rock foundation, whereas, the one which did not exhibit cracking was in the order of 0.5 H above rock foundation. It can be illustrated how tensile cracking could occur in this lift. In this illustration a compressive strength of the concrete at the time of cracking of 5000 psi, a tensile strength of 10 per cent of the compressive strength (or 500 psi), a modulus of elasticity of 5×10^6, a coefficient of expansion and contraction[10] of 5.5×10^{-6}, a maximum temperature differential of 70° F (to cool from 125 to 55° F, the mean annual temperature for the region) are used. Then utilizing the simple formula $S = Ee$, where "S" is the tensile stress to which the concrete is subjected in psi, "E" is the modulus of elasticity of the concrete in psi and "e" is the strain to which the concrete is subjected in in/in, the tensile stress to which the concrete in this lift could be subjected, neglecting the effect of plastic flow and assuming full restraint, would be in the order of 1925 psi. If plastic flow were accounted for by reducing the restraint effect from 100 per cent at the base of the dam to zero per cent at 0.5 H above the base as has been practiced by some investigators, the stress to which this block would be subjected (utilizing a working modulus of elasticity of 3×10^6) would be in the order of 1100 psi. Both of the above assumptions would indicate tensile stresses to which the lift showing cracking could have been subjected that would exceed the tensile strength of the concrete. The other lift to which it was compared having equally high temperature differentials would not have been subjected to these high stresses, if it is assumed that the effect of plastic flow increases to 100 per cent and the restraint effect to zero at 0.5 H above the base.

It is felt that the relatively rapid cycle of heat generation and dissipation accompanied by the development of high compressive strength in the concrete at a relatively early age (28 days) were important factors in the reduction of the tendency for cracking at North Fork. This rapid cycle of heat generation and dissipation would permit plastic flow to occur while the concrete was relatively young and elastic thereby permitting the application of temperature contraction stresses with absence of the development of high accompanying resisting tensile stresses. In addition the relatively high strength of the concrete at an early age would tend to assure the existence of adequate potential resisting strength to resist cracking.

Heat Generation in Spillway and Intake Dam Concrete, Noxon Rapids

Eighteen Carlson Type MA, resistance thermometers were embedded in the interior of massive lifts of concrete in the spillway and intake dam at Noxon Rapids to determine the degree of heat generation developed under varying conditions of mass, weather, class of concrete (cement content), and distance between specific parts of the mass being observed.

To reduce the initial placed temperature of the concrete during warm weather the coolest water and aggregates without resorting to artificial cooling media were utilized. Mixing and curing water was obtained from a mountain creek, and coarse aggregates were cooled by evaporation by sprinkling in the stockpiles and rinsing on the re-screen. No ice was utilized in the mixed concrete.

To reduce the maximum temperature generated by the mass concrete in place during warm weather, specification and design requirements enumerated below were utilized.

(1) Thin lift heights of 5 feet were utilized as the basic lift height in massive structures throughout the construction period to aid in the dissipation of internal heat. In addition the first two lifts off most foundation rock and off specific lifts of old concrete were placed in 2-1/2 ft thick courses. These 2-1/2 ft lifts served to reduce maximum heat generation and subsequent stress concentration in locations of high restraint at rock foundation and at older concrete masses.

(2) The time between successive lifts of concrete was extended to as long as 96 hours to permit a period of time for heat dissipation from each lift of concrete before it was covered by additional insulating lifts of concrete.

(3) Continuous water spray and ponding was utilized on the horizontal construction joints for cooling the concrete, and a gray-pigmented curing compound was utilized on vertical surfaces for improved heat reflecting properties.

(4) Lean interior concrete having cement contents as low as 2.25 and 2.00 sk/cu yd, and face concrete having a relatively low cement content of 3.75 sk/cu yd were utilized to reduce the heat of hydration potential of the concrete in place.

The results of temperature observations obtained from the eighteen thermometers embedded in the concrete at Noxon are shown in Tables 4A, B, C, and D and in Figs. 3A, B, C, and D. Sixteen of the thermometers were embedded in the interior mass concrete, and two were embedded in the exterior face concrete, one thermometer being situated within 5 feet of the outside face surface and the other within 2.5 feet. Ten of the thermometers were placed during summer weather, and four more were placed during fall weather. Two of the thermometers were placed in successive lifts of concrete.

TABLE 4A - HEAT GENERATION IN MA S CONCRETE PLACED IN 5 FT LIFTS UNDER VARYING CONDITIONS OF MASS AS OBSERVED BY MEANS OF CARLSON TYPE MA RESISTANCE THERMOMETERS - NOXON RAPIDS HYDRO ELECTRIC DEVELOPMENT

Heat Generation in the Interior of Concrete Lifts Having Physical Characteristics Indicated, °F - Effect of Mass of Section			
Time After Placement (Days)	Therm Placed 2.5 Ft From D/S Face: 3.75 sk/cu yd; Placed in Summer: (5 x 50 Ft Sec); Th No T14	Therm Placed 5 Ft From U/S Face; 3.75 sk/cu yd; Placed in Summer; (10 x 50 Ft Sec); Th No T12	Therm Placed in Interior Mass; 2.5 sk/cu yd; Concrete placed in Summer; (50 x 70 Ft Sec); Th No T13
0	61	61	59
1	82.7	--	74.6
2	81.4	91.9	75.4
4	82.7	93.2	79.4
6	--	96.4	84.9
8	82.2	102.5	86.7
10	81.1	102.5	88.6
20	67.9	101.3	93.1
30 (1 mo)	62.7	96.2	95.3
60 (2 mo)	47.8	80.3	94.4
90 (3 mo)	49.3	76.9	91.2
180 (6 mo)	47.0	66.2	81.0
270 (9 mo)	57.1	61.9	72.0
365 (1 yr)	57.4	61.0	66.5
Max Temp Incr °F	21.7	43.5	36.3
Mixing Water Temp Range, °Fs	(59 Mean) 58-60	(59 Mean) 58-60	(59 Mean) 58-60
Air Temp Range, °Fs	58-28 (43 Mean)	58-28 (43 Mean)	58-28 (43 Mean)
Coef of Max Temp Incr °F/sk	8.7	11.1	14.5
% of Total Conc Surf Area Exposed During Observ Period**	9-100%	9-100%	9-100%

Notes: * 1. During day of placement of concrete.
**2. Total potential exposed surface is summation of areas (1) upstream and downstream faces, (2) two sides and (3) top horizontal construction joints. For Complete Summary see Table 6.

All of the thermometers were embedded in the center of the mass of the lift and class of concrete being observed. Thirteen of the thermometers were embedded at mid-depth in the 5 ft lifts and five of the thermometers were embedded at the rock-concrete contact plane at the bottom of two 5 ft lifts. The distances of the thermometers from the upstream and downstream faces ranged from 2-1/2 to 5 ft for those embedded in Class A (face) concrete and from 32 to 60 ft for those embedded in the Class B (interior) concrete. The distances of the thermometers from the side vertical construction joints of the blocks ranged from 24 to 38 ft. Maximum exposure of concrete surfaces from which heat could be dissipated in lifts observed at Noxon occurred at 1 to 3 days after placement when all forms were stripped and the duration of the exposure extended over a period ranging from 1 to 134 days, the usual period being from 1 to 3 days. Minimum exposure of surfaces permitting heat dissipation occurred at from 10 to 355 days after placement of the lift being observed when additional placed lifts contained all but upstream and downstream face concrete. The percentage of the total potential concrete surface area from which heat could be dissipated from the class of concrete being observed throughout the entire observation periods ranged from a maximum of 100 per cent to a minimum of 3 per cent. Tabulations of surface exposure conditions for all lifts on which interior temperatures were observed at Noxon are contained in Table 6.

The highest recorded internal temperature attained in a specific lift of concrete at Noxon was 102.5° F. This temperature was attained in the upstream 10 ft thick section of face concrete having a cement content of 3.75 sk/cu yd. The concrete was placed during summer weather at an initial temperature of 61° F. The lowest recorded maximum internal temperature attained in specific lift of concrete was 82.7° F. This temperature was attained in a downstream 5 ft thick face section of concrete, and the recording thermometer was placed in the center of the section at a distance of 2.5 feet from the face. The concrete in this section had a cement content of 3.75 sk/cu yd and was

TABLE 4B - HEAT GENERATION IN MASS CONCRETE PLACED IN 5 FT LIFTS UNDER VARYING CONDITIONS OF CLASS (CEMENT CONTENT) AS OBSERVED BY MEANS OF CARLSON TYPE MA RESISTANCE THERMOMETERS - NOXON RAPIDS HYDRO ELECTRIC DEVELOPMENT

Heat Generation in the Interior of Concrete Lifts Having Physical Characteristics Indicated, °F Effect of Class (Cement Content) of Concrete			
Time After Placement (Days)	Class A, 3.75 sk/cu yd; Placed in Summer; (10 x 50 Ft Sec); Th No T12	Class B, 2.5 sk/cu yd; Placed in Summer; (70 x 50 Ft Sec); Th No T9, 10, 11, 13 (Av)	Class A, 2.0 sk/cu yd; Placed in Summer; (50-70 x 120-140 Ft Sec); Th No T5, 6, 7, 8 (Av)
0	61	59.8	60.8
1	—	77.5	66.7
2	91.9	79.5	73.1
4	93.2	84.4	70.5
6	96.4	89.0	79.2
8	102.5	91.7	80.9
10	102.5	91.7	82.2
20	101.3	94.3	85.2
30 (1 mo)	96.2	95.1	85.4
60 (2 mo)	80.3	86.8	83.9
90 (3 mo)	76.9	83.8	79.1
180 (6 mo)	66.2	74.7	71.1
270 (9 mo)	61.9	67.9	66.1
365 (1 yr)	61.0	64.6	62.6
Max Temp Incr °F	41.5	35.3	24.6
Mixing Water Temp Range, °F*	(59 Mean) 58-60	(59 Mean) 58-60	(54 Mean) 50-58
Air Temp Range, °F*	58-28 (43 Mean)	58-28 (43 Mean)	54-40 (47 Mean)
Coef of Max Temp Incr °F/sk	11.1	14.1	12.3
% of Total Conc Surf Area Exposed During Observ Period**	9-100%	8-100%	3-77%

Notes: * 1. During day of placement of concrete.
**2. Total potential exposed surface is summation of areas (1) upstream and downstream faces (2) two sides and (3) top horizontal construction joints. For Complete Summary see Table 6.

placed during summer weather at an initial temperature of 61° F. Other low internal temperatures averaging near 85° were attained in lifts of lean fill concrete used to fill a deep hole under the intake. This concrete had a cement content of 2.0 sk/cu yd and was placed during summer weather at an average initial temperature of about 61° F. The total temperature rise in the mass concrete at Noxon resulting from internal heat generation regardless of section size, weather conditions, cement content, or surface exposure conditions, ranged from 20.5 to 41.5° F. When this temperature development is analyzed on the basis of design cement content of the concrete placed, the "coefficient" for maximum temperature rise at Noxon then becomes 7.5 to 15.3° F rise/sk of cement.

In Table 4A and Fig. 3A the effect of the mass of the section upon interior heat generation is shown. The thermometer embedded in the concrete at a distance of 5 ft from the face registered a temperature increase of nearly twice as much as a thermometer embedded under similar conditions in the same lift and in concrete having the same cement content but embedded at a distance of 2.5 ft from the face. A third thermometer embedded in the interior of the same block in lean concrete registered nearly as great an increase in temperature as the thermometer in the richer face concrete 5 ft from the face and a greater increase than the thermometer in the richer face concrete 2.5 ft from the face.

The effect of class of concrete or cement content is shown in Table 4B and in Fig. 3B. In this comparison, it is shown that for Noxon, the maximum interior temperature rise for different classes of concrete placed under similar conditions increased directly with the cement content of the mix, ranging from an increase of 24.6° F for the 2.0 sk/cu yd mix, to 35.3° F for the 2.5 sk/cu yd mix, to 41.5° F for the 3.75 sk/cu yd mix.

The effect of weather conditions during placement of the concrete on the total temperature rise of the concrete is shown in Table 4C and Fig. 3C. This comparison shows that for similar conditions of mass and cement content at

TABLE 4C - HEAT GENERATION IN MASS CONCRETE PLACED IN 5 FT LIFTS UNDER VARYING CONDITIONS OF WEATHER AS OBSERVED BY MEANS OF CARLSON TYPE MA RESISTANCE THERMOMETERS, NOXON RAPIDS HYDRO ELECTRIC DEVELOPMENT

Time After Placement (Days)	Heat Generation In Interior Mass Concrete Under Conditions Indicated, °F. Effect of Varying Weather Conditions During Placement. Placed in Summer: 2.5 sk/cu yd 70 x 50 Ft Sec; Th No T9,10,11, 13 (Av)	Placed in Spring; 2.5 sk/cu yd; (50-70) x 70 Ft Sec; Th No T1,2 3, 4 (Av)	Placed in Fall: 2.5 sk/cu yd; (40-50) x (50 -75) Ft Sec; Th No T15,16,17(Av)	Placed in Fall; 3.75 sk/cu yd; 25 x 75 Ft Sec; Th No T18
0 (At Placement)	59.8	47.9	53.7	52.0
1	77.5	59.6	58.3	66.6
2	79.5	62.4	58.6	69.3
4	84.4	65.2	57.2	--
5	85.1	--	--	78.4
7	87.8	69.9	--	80.1
8	91.7	71.0	69.5	--
15	93.4	75.3	--	--
20	94.3	77.0	73.2	76.6
30 (1 mo)	95.1	78.8	70.2	73.6
60 (2 mo)	86.8	81.0	--	--
90 (3 mo)	83.8	81.2	58.3	60.7
180 (6 mo)	74.7	78.5	58.6	55.1
270 (9 mo)	67.9	72.0	62.2	61.3
365 (1 yr)	64.6	62.8	62.7	59.3
Max Temp Increase °F	35.3	33.3	20.5	28.1
Mixing Water Temp Range, °Fs	(59 Mean) 58-60	(48 Mean) 42-54	(54 Mean) 50-58	(54 Mean) 50-58
Air Temp Range °Fs	(53 Mean) 80-26	(40.5 Mean) 57-24	(39 Mean) 48-30	(44 Mean) 50-38
Coef of Max Temp Increase °F/sk of Cem	14.1	13.3	8.2	7.5
% of Total Conc Surf Area Exposed During Observ Period**	8-100%	8-100%	7-100%	7-100%

Notes: * 1. During day of placement of concrete.
**2. Total potential exposed surface is summation of areas (1) upstream and downstream faces, (2) two sides and (3) top horizontal construction joints. For Complete Summary see Table 6.

Noxon, concrete placed in the summer and spring increased in temperature a significant degree more (in the order of 5 to 15° F) than concrete placed in the fall. In comparing a series of similar lifts of concrete placed in the summer and fall, one series in which the temperature increase of 2.5 sk/cu yd interior concrete was observed and the other in which the temperature increase of 3.75 sk/cu yd interior was observed, it is significant to note that the lean concrete placed in the summer showed a greater increase than that of the richer concrete placed in the fall. In this case the effect of weather on the interior temperature increase of the concrete was greater than that of cement content.

The temperature differential between successive lifts of concrete placed four days apart is shown in Table 4D and Fig. 3D. This data shows that for comparable conditions of weather, cement content, and mass, two successive lifts of concrete will attain the same approximate internal temperature within

TABLE 4D - HEAT GENERATION DIFFERENTIAL IN TWO SUCCESSIVE LIFTS OF MASS CONCRETE PLACED IN 5 FT LIFTS AS OBSERVED BY MEANS OF CARLSON TYPE MA RESISTANCE THERMOMETERS
NOXON RAPIDS HYDRO ELECTRIC DEVELOPMENT

Heat Generation in Interior Mass Concrete Under Conditions Indicated, °F Differential in Temperature Between Successive Lifts		
Time After Placement (Days)	Lift El 2235-2240: Placed in Spring; 2.5 sk/cu yd; 55 x 70 Ft Sec; Th No T3	Lift El 2240-2245; Placed in Spring 4 days after lift El 2235-2240; 2.5 sk/cu yd; 50 x 70 Ft; Th No 4
0 (At Placement)	50.3	Not Placed
1	64.1	Not Placed
2	--	Not Placed
4	70.3	50.9 (At Placement)
5	70.6	61.3
7	72.8	65.8
8	73.9	67.2
15	78.5	77.5
20	80.4	80.4
30 (1 mo)	82.3	85.5
60 (2 mo)	86.2	88.7
90 (3 mo)	86.3	89.1
180 (6 mo)	82.4	84.3
270 (9 mo)	75.5	69.5
365 (1 yr)	61.5	59.1
Max Temp Increase °F	36.0	38.2
Mixing Water Temp Range, °F*	(48 Mean) 42-54	(48 Mean) 42-54
Air Temp Range °F*	(38.5 Mean) 57-30	(43 Mean) 54-32
Coef of Max Temp Increase °F/sk of Cem	14.4	15.3
% of Total Conc Surf Area Exposed During Observ Period**	10-100%	11-100%

Notes: * 1. During day of placement of concrete.

**2. Total potential exposed surface is summation of areas (1) upstream and downstream faces, (2) two sides and (3) top horizontal construction joints. For Complete Summary see Table 6.

a period of about two weeks after placement of the second lift. This comparison only serves to show the effect of one section of mass upon another in contact with it, the masses having a relatively large difference in internal temperature initially, but both attaining relatively the same temperature a short time after contact and maintaining this close relationship in temperature for the remainder of the observation period.

The cycle of maximum heat generation and dissipation in the concrete at Noxon was similar but somewhat slower than that of North Fork. From about 75 to 100 per cent of the maximum temperature had been attained in 48 hr (2 days) and from about 85 to 100 per cent had been attained in 120 hr (5 days) after placement. The maximum temperature had been attained and had begun to decrease in most lifts by 20 to 30 days after placement and had begun to decline in all lifts by 90 days after placement. The weather and size of section had a significant effect upon the time in which a reduction in maximum temperature began. Interior concrete placed in the summer and spring had attained its maximum temperature and begun to cool in a period of 1 to 3 months after placement. Interior concrete placed in the fall and concrete located near the outside face (2.5 to 5 ft from the surface) had attained its maximum temperature and started to cool within one to weeks after placement. The only mass concrete at Noxon which had observed interior temperatures near or at the mean annual temperature for the region (45° F) within a period of 9 months after placement was that located in the 5 ft face section described previously.

It is significant to note that the initial placed concrete temperature at Noxon very nearly approached that of the mixing water used.

As a guide for future projects having concrete sections comparable with those at Noxon (lean interior sections 5 ft high and in the order of 35-50 x 75-125 in cross section and richer face sections 5 feet high, 5 and 10 feet thick and in the order of 35 to 50 feet wide) it appears that the following table of coefficients could be utilized for estimating the maximum internal temperature increase to be expected under varying field conditions:

	Concrete Placed in Summer or Spring	Concrete Placed in Fall
Interior Mass	12 to 15° F/sk	7 to 8° F/sk
Exterior Face	9 to 11° F/sk	5 to 6° F/sk

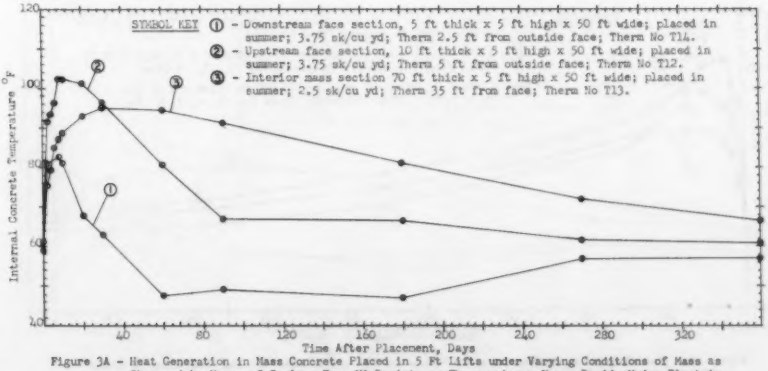
SYMBOL KEY
① - Downstream face section, 5 ft thick x 5 ft high x 50 ft wide; placed in summer; 3.75 sk/cu yd; Therm 2.5 ft from outside face; Therm No T14.
② - Upstream face section, 10 ft thick x 5 ft high x 50 ft wide; placed in summer; 3.75 sk/cu yd; Therm 5 ft from outside face; Therm No T12.
③ - Interior mass section 70 ft thick x 5 ft high x 50 ft wide; placed in summer; 2.5 sk/cu yd; Therm 35 ft from face; Therm No T13.
Internal Concrete Temperature °F
120
100
80
60
40
40
80
120
160
200
240
280
320
Time After Placement, Days

Figure 3A - Heat Generation in Mass Concrete Placed in 5 Ft Lifts under Varying Conditions of Mass as Observed by Means of Carlson Type MA Resistance Thermometers, Noxon Rapids Hydro-Electric Development

The comparison would have to be made for those projects utilizing Type II cements and utilizing no embedded cooling coils as was the case at Noxon.

The mean coefficient for maximum internal temperature increase in concrete at Noxon regardless of size of section, weather conditions during placement, and cement content was 11.4° F/sk.

Cracking which could have been attributable to heat generation and subsequent cooling occurred at Noxon principally in lean interior massive sections of the spillway and intake dam having compressive strengths in the order of 2000 psi at 28 days and located at a height of about 0.3H above the rock foundation. The concrete exhibiting this cracking had initial temperatures ranging from 55-70° F and was placed in late spring and summer weather. The particular sections showing cracking unfortunately had no resistance thermometers embedded within them. However, based on the results shown in Tables 4A through D and Figs. 3A through D, this concrete which contained cement contents of 2.5 sk/cu yd in the interior and 3.73 sk/cu yd in the face, would have been expected to have attained estimated internal temperatures ranging from about 90-105° F in the interior mass sections, from about 75-90° F in the downstream face sections and from 95-110° F in the upstream face sections. It can be illustrated how cracking could occur in the concrete at these temperature differentials. For the lean concrete using a potential resisting tensile stress of 10 per cent for a compressive strength of 2000 psi at the time of cracking (or 200 psi) a modulus of elasticity of 2×10^6 psi, a coefficient of thermal expansion and contraction of 5.5×10^{-6} in/in per degree F, a maximum temperature reduction of 50° F using the formula S = Ee, and using 100 per cent restraint to movement or no plastic flow, the applied stress would equal about 690 psi and would exceed the internal resisting stress. Assuming that plastic flow at 0.30 H above the base would reduce the restraint to 40 per cent and utilizing a working modulus of elasticity of 0.8×10^6 the applied stress could have been in the order of 220 psi, and the possibility of cracking would still exist. By using a maximum temperature reduction of 55° F for the upstream face sections, a compressive strength of 5000 psi, a tensile strength of 500 psi, a working modulus of elasticity of 2×10^6, the applied stress would have been in the order of 600 psi, and the possibility for cracking in the upstream face concrete would also exist. The downstream face

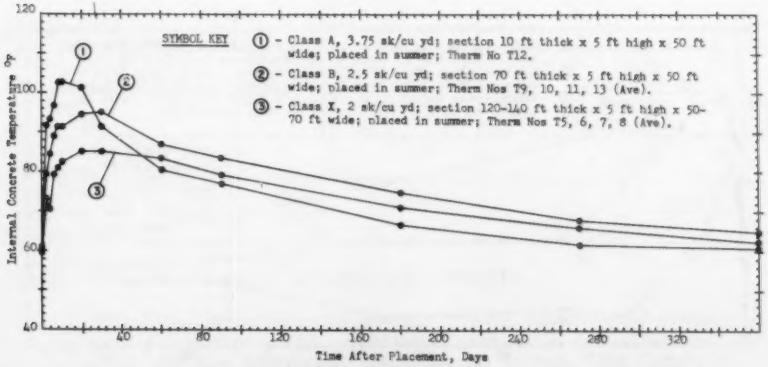
SYMBOL KEY
① - Class A, 3.75 sk/cu yd; section 10 ft thick x 5 ft high x 50 ft wide; placed in summer; Therm No T12.
② - Class B, 2.5 sk/cu yd; section 70 ft thick x 5 ft high x 50 ft wide; placed in summer; Therm Nos T9, 10, 11, 13 (Ave).
③ - Class X, 2 sk/cu yd; section 120-140 ft thick x 5 ft high x 50-70 ft wide; placed in summer; Therm Nos T5, 6, 7, 8 (Ave).
Internal Concrete Temperature °F
120
100
80
60
40
40
80
120
160
200
240
280
320
Time After Placement, Days

Figure 3B - Heat Generation in Mass Concrete Placed in 5 Ft Lifts Under Varying Conditions of Class (Cement Content) of Concrete as Observed by Means of Carlson Type MA Resistance Thermometers, Noxon Rapids Hydro Electric Development

sections having a lower maximum temperature reduction (about 40° F) would show on this analysis to be less susceptible to cracking having a resisting stress of about 500 psi and an applied stress of about 450 psi.

Comparison of Heat Generation, North Fork and Noxon

The sections observed by means of Carlson resistance thermometers at North Fork were all relatively thin, the blocks being about 50 feet wide (along dam axis) and varying from about 15 to 35 feet thick to 5 to 10 feet high. Interior mass sections at Noxon were all relatively large varying from about 50 to 75 feet wide (along dam axis) to 25 to 140 feet thick to 5 feet high. Face sections at Noxon were thin ranging from about 50 feet wide (along dam axis) to 5 to 10 feet thick to 5 feet high. The concrete in the North Fork arch dam was all of relatively high cement content, varying from 4 to 4.75 sk/cu yd, while the concrete at Noxon was made up of a relatively high cement content concrete at the face (of 3.75 sk/cu yd) and lean concrete in the interior mass (of 2 to 2.5 sk/cu yd.)

It is significant to note that even though the maximum temperature generation in the concrete at North Fork was greater than that at Noxon, (North Fork having a temperature increase range of about 30 to 60° F and Noxon, 20 to 40° F) the "Coefficient of Maximum Temperature Increase" based on the design cement content of the concrete were comparable, North Fork's coefficient ranging from 6.1 to 14.2° F/sk (Mean 10.5) and Noxon's ranging from 7.5 to 15.3° F/sk (Mean 11.4).

In comparing the temperature generation trends of North Fork and Noxon by size of section, a significant increase in temperature generation was shown at Noxon in the 10 ft thick face section as compared with the 5 ft face section, and at North Fork in the 10 ft lift height as compared with the 5 ft lift height. In sections larger than 10 ft in section thickness no significant difference in temperature generation potential was noted with additional increase in size on either project.

The same trend for internal temperature to increase more in those sections placed in the spring and summer as compared with those placed in the fall and winter was shown on both projects.

The period of time required for the maximum internal temperature to be attained in the concrete and for the concrete to begin to cool was similar for

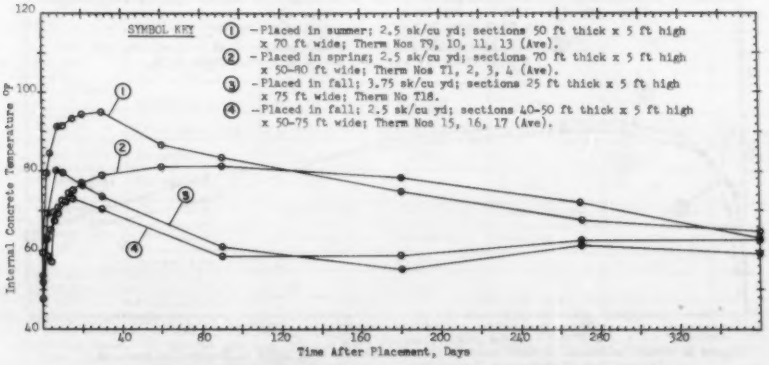
SYMBOL KEY
① – Placed in summer; 2.5 sk/cu yd; sections 50 ft thick x 5 ft high x 70 ft wide; Therm Nos 79, 10, 11, 13 (Ave).
② – Placed in spring; 2.5 sk/cu yd; sections 70 ft thick x 5 ft high x 50-80 ft wide; Therm Nos T1, 2, 3, 4 (Ave).
③ – Placed in fall; 3.75 sk/cu yd; sections 25 ft thick x 5 ft high x 75 ft wide; Therm No T18.
④ – Placed in fall; 2.5 sk/cu yd; sections 40-50 ft thick x 5 ft high x 50-75 ft wide; Therm Nos 15, 16, 17 (Ave).
Internal Concrete Temperature
Time After Placement, Days
120
100
80
60
40
40
80
120
160
200
240
280
320

Figure 3C – Heat Generation in Mass Concrete Placed in 5 Ft Lifts under Varying Conditions of Weather During Placement as Observed by Means of Carlson, Type MA, Resistance Thermometers, Noxon Rapids Hydro Electric Development

both projects but somewhat slower for Noxon. The time required for the interior concrete temperatures to approach the mean annual temperature of the region was significantly longer for Noxon. The percentage of surface area available to dissipate internal heat throughout the observation period was significantly more and time intervals between placing of subsequent lifts were longer for lifts observed at North Fork than for those observed at Noxon. This condition coupled with the large difference in massiveness of sections observed on the two projects undoubtedly accounted for the slower dissipation of internal heat at Noxon. A significant proportion of the maximum internal temperature of the blocks on both projects was attained in a period of 48 hrs after placement (75 to 100%). This trend for early maximum heat generation coupled with the tendency for the internal temperature of any specific block to attain an equilibrium temperature with other blocks in contact with it (above or below) show the benefit to be derived from permitting the longest practical interval of time between successive lifts of concrete. By permitting a period of time for heat generation and dissipation to occur between successive lifts of concrete, the heat generation potential of each lift is reduced before the surfaces which are dissipating heat are contained by an additional insulating lift of concrete.

The effect for an increase of cement content to result in an increase in the total internal temperature generated in the concrete was shown on both projects.

Whereas, the initial temperature of the concrete at the time of placing compared more closely with that of the current mean air temperature of North Fork, it compared more closely to the mixing water temperature at Noxon. The trend for the initial concrete temperature to follow the trend of both the mean air temperature and the mixing water temperature (in the absence of utilization of artificial means to lower the initial temperature of the concrete) was, however, similar on both projects.

The effect of compressive strength upon the cracking tendency of concrete with volume change is not definitely defined by this data, but it does seem to indicate a trend. It is significant to note that cracking at Noxon occurred principally in lean interior massive sections having ultimate compressive strengths in the order of 2000 to 3000 psi when the maximum internal temperature cycle of increase and decrease was in the order of 20 to 40° F. However,

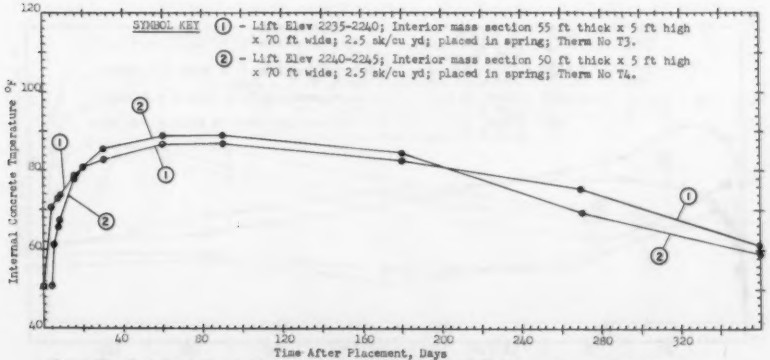
SYMBOL KEY
① - Lift Elev 2235-2240; Interior mass section 55 ft thick x 5 ft high x 70 ft wide; 2.5 sk/cu yd; placed in spring; Therm No T3.
② - Lift Elev 2240-2245; Interior mass section 50 ft thick x 5 ft high x 70 ft wide; 2.5 sk/cu yd; placed in spring; Therm No T4.
Internal Concrete Temperature °F
120
100
80
60
40
40
80
120
160
200
240
280
320
Time After Placement, Days

Figure 30 - Heat Generation in Mass Concrete in Two Successive 5 Ft Lifts as Observed by Means of Carlson Type HA Resistance Thermometers - Noxon Rapids Hydro-Electric Development

significantly less cracking was noted at North Fork and Noxon in the richer concrete sections having ultimate compressive strengths in excess of 5000 psi when the maximum internal temperature cycle of increase and decrease was as high as 40 to 70° F. Regardless of this apparent trend, it does not appear possible to absolutely control cracking in massive structures by the control of strength alone without the use of artificial methods of cooling the concrete either during the mixing and placing stage or afterward when the concrete is in place by cooling coils or by combinations of both methods. The basic fact which defeats any attempt to control cracking by controlling strength alone is that as the strength requirement increases, the cement requirement also increases and as the cement content increases, the total heat generation and thus the cracking tendency increases. Additional data other than that obtained in this investigation are required to definitely establish the limits of this interrelationship of strength—heat generation and cracking tendency.

That it is necessary to utilize methods and equipment to positively prevent all cracking resulting from heat generation and dissipation volume change particularly in interior sections is not within the scope of this paper. However, in the author's opinion it is a subject worthy of important discussion by both design and field control Engineers.

Project Design and Supervision

The North Fork Hydro-electric Development was constructed for the Portland General Electric Company of Portland, Oregon and the Noxon Rapids Hydro-electric Development is being constructed for the Washington Water Power Company of Spokane, Washington. General O. E. Walsh, USA (Ret) is Vice President in charge of Engineering for the Portland General Electric Company, and K. O. Strenge is Chief Civil Engineer for The Washington Water Power Company. Design and construction management for both projects was by Ebasco Services Incorporated of New York. J. M. Mullarkey was the

TABLE 5 - PERCENTAGES OF TOTAL POTENTIAL CONCRETE SURFACE AREA EXPOSED TO THE OUTSIDE AIR FOR LIFTS OF CONCRETE ON WHICH INTERNAL TEMPERATURES WERE OBSERVED UTILIZING CARLSON TYPE MA RESISTANCE THERMOMETERS
NORTH FORK HYDRO ELECTRIC DEVELOPMENT

Conditions of Concrete Surface Exposure	Time Conditions Existed and Exposed Areas (Ranges) Days, %	Thermometer Identification (Reference Table 3)				
		Therm No 1A, 2B, 12A	Therm No 3A, 4B, 5B	Therm No 7A, 9A	Therm No 2A, 4A, 5A	Therm No 7B - 7A
Concrete placed: forms in place: top horizontal construction joint (only) exposed.	After placed, Da	0 to (2-3)	0 to (2-3)	0 to (2-3)	0 to (2-3)	0 to 3
	Interval, Da	1 to 3	1 to 3	1 to 3	1 to 3	3
	Exposed Area, %	38 - 39	43 - 45	44 - 49	62 - 67	41
All forms stripped: upstream and downstream faces, top horizontal constr joint and all exposed ends open.	After placed, Da	(2-3)to(6-34)	(2-3)to(3-7)	(2-3)to(14-46)	(2-3)to(3-11)	2 to 6
	Interval, Da	3 to 32	1 to 5	11 to 42	1 to 9	4
	Exposed Area, %	88 - 93	38 - 100	31 - 100	93 - 94	91
Horizontal construction joint of lift being observed covered with additional lift: upstream and downstream faces and exposed ends open.	After placed, Da	(6-34)to(13-50)	(3-7)to(6-11)	(14-46)to(14-72)	(3-11)to(5-23)	6 to End
	Interval, Da	7 to 16	0 to 8	0 to 26	0 to 22	Unlimited
	Exposed Area, %	49 - 55	43 - 48	31 - 56	26 - 32	77
All surfaces except upstream and downstream faces covered by additional lifts of concrete.	After placed, Da	(13-50)to End	(6-11) to End	(14-72) to End	(5-23) to End	-
	Interval, Da	Unlimited	Unlimited	Unlimited	Unlimited	Unlimited
	Exposed Area, %	41 - 47	37 - 41	31 - 38	19 - 26	77
Max %, Total potential surface area exposed during observation period.		93	100	100	94	91
Min %, Total potential surface area exposed during observation period		38	37	31	19	41

Note: The total potential exposed surface area is considered to be the summation of the areas of (1) the upstream and downstream faces, (2) the two ends and (3) the top exposed horizontal construction joint surfaces of the lifts on which internal temperatures were observed.

Project Design Engineer for North Fork, and R. A. Sutherland is the Project Design Engineer for Noxon Rapids. F. C. Schlemmer is Construction Manager for both projects. F. L. Weiss is the Project Manager for Noxon Rapids, and Roger P. O'Reilly was the Construction Superintendent for North Fork. The author organized and supervised the concrete inspection and control programs for the two projects.

ACKNOWLEDGEMENT

The author is indebted to Mr. H. K. Fairbanks, Consulting Civil Engineer for Ebasco Services Incorporated, for constructive criticism and suggestions made on first drafts of this report, thereby aiding in its preparation for final form.

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TABLE 6 - PERCENTAGES OF TOTAL POTENTIAL CONCRETE SURFACE AREA EXPOSED TO THE OUTSIDE AIR FOR LIFTS OF CONCRETE ON WHICH INTERNAL TEMPERATURES WERE OBSERVED UTILIZING CARLSON TYPE MA RESISTANCE THERMOMETERS
NOXON RAPIDS HYDRO ELECTRIC DEVELOPMENT

Conditions of Concrete Surface Exposure	Time Conditions Existed and Exposed Areas (Ranges) Days, %	Thermometer Identifications (Reference Tables 4A, B, C & D)					
		Therm No 1, 2, 3 & 4	Therm No 5,6,7 & 8	Therm No 9,10,11 & 13	Therm No 12,13,14	Therm No 15, 16 & 17	Therm No 18
Concrete placed: forms in place: top horizontal constr joint (only) exposed.	After placed, Da	0 to (1-3)	0 to 1	0 to (1-3)	0 to (1-3)	0 to (1-3)	0 - 2
	Interval, Da	1 - 3	1	1 - 3	1 - 3	1 - 3	2
	Exposed Area, %	78 - 80	77	76 - 77	76	77 - 81	81
All forms stripped: upstream and downstream faces, top horizontal constr joint and all exposed ends open.	After placed, Da	(1-3)to(3-4)	1 to 2	(1-3) to 4	(1-3) to 4	(1-3)to(6-135)	2 - 4
	Interval, Da	1 - 3	1	1 to 3	1 to 3	3 to 134	4
	Exposed Area, %	100	77	100	100	100	100
Horizontal constr joint of lift being observed covered with additional lifts: upstream and downstream faces and exposed sides open.	After placed, Da	(3-4)to(10-355)	2 to 3	4 to (15-34)	4 to 13	(6-135)to(17-142)	4 to (17-27)
	Interval, Da	6 to 351	1	11 to 30	9	7 to 21	13 to 23
	Exposed Area, %	14 - 22	3	23 - 24	24	19 - 23	13 - 19
All surfaces except upstream and downstream faces covered by additional lifts of concrete.	After placed, Da	(10-355) to End	3 to End	(15-34)to End	13 to End	(17-142) to End	(17-27)to End
	Interval, Da	Unlimited	Unlimited	Unlimited	Unlimited	Unlimited	Unlimited
	Exposed Area, %	8 - 11	3	8 - 9	9	7 - 12	7
Max %, Total potential surface area exposed during observation period.		100	77	100	100	100	100
Min %, Total potential surface area exposed during observation period		8	3	8	9	7	7

Note: The total potential exposed surface area is considered to be the summation of the areas of (1) the upstream and downstream faces, (2) the two ends, and (3) the top exposed horizontal construction joint surfaces of the lifts on which internal temperatures were observed.

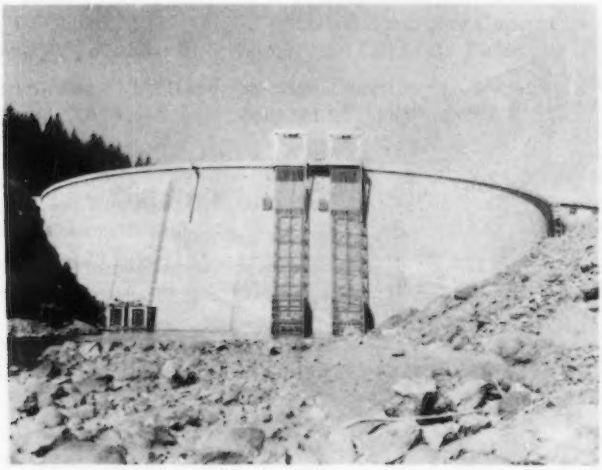

Figure 4 - North Fork Hydro-Electric Development, Located on the Clackamas River Near Portland, Oregon. Upper Photograph Shows Upstream Face of Arch Dam. Lower Photograph Shows Downstream Face of Arch Dam, Powerhouse, Spillway, and Fish Passing Facility Structures Under Construction.

Figure 5 - Noxon Rapids Hydro-Electric Development Dam and Powerhouse, Located on Clark Fork River in Northwestern Montana Equidistant Between Spokane, Washington and Missoula, Montana. Upper Photograph Shows Dam, Looking Downstream. Lower Photograph Shows Dam and Powerhouse, Under Construction, Looking Upstream.

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Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

RAISING TRANSMISSION TOWERS WITH ENERGIZED LINES

Albert G. Masters[1] and Ernest J. Gesing[2]

ABSTRACT

Demand for power makes it necessary for The Illuminating Company to build additional lines and also to reconductor existing lines with larger wires. Towers on existing lines are increased in height by adding an extension to the bottom by this method to maintain code ground clearance under new and larger conductors.

The increasing demand for power makes it necessary for The Illuminating Company to build additional lines and also to increase the capacity of existing lines by reconductoring with larger wires.

When the decision has been made to reconductor an existing line, some of the towers must be increased in height to maintain code ground clearances under the new and larger conductors because of the increased sag.

There are several methods in use for raising towers. None of them are simple and all are expensive. In most cases the lines must be de-energized and the work done on week ends with special mobile-rented equipment.

The method recently developed by The Illuminating Company and presently used for tower extension work will be discussed. It was found to be most successful. Manhours run about 250 per tower extension and will decrease as the crews become more experienced. All material is completely re-useable and standard line trucks and equipment are used. Lines remain energized unless it is convenient to de-energize them and the work can be done without critical scheduling. As a safety precaution towers are not raised when the wind exceeds 15 miles per hour normal to the line.

Note: Discussion open until March 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2208 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 5, October, 1959.

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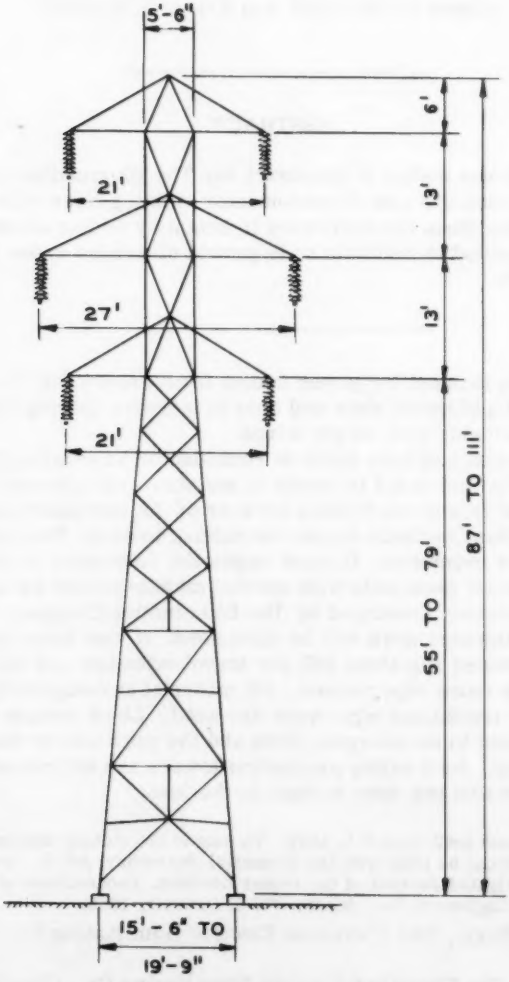
2. Senior Engr., The Cleveland Electric Illuminating Co., Cleveland, Ohio.

Description of Towers

The transmission towers are part of The Illuminating Company's 132 KV transmission system; see Figs. 1 and 2. They are of the double circuit variety, with the bottom crossarm from fifty to eighty feet above grade. The crossarms are spaced thirteen feet apart and the ground wire attachment is six or eleven feet above the top crossarm. These towers vary in over-all height from ninety to one-hundred and fifteen feet. They are five and one-half feet square at the top and the bases vary from fifteen and one-half feet to twenty feet square at the ground line.

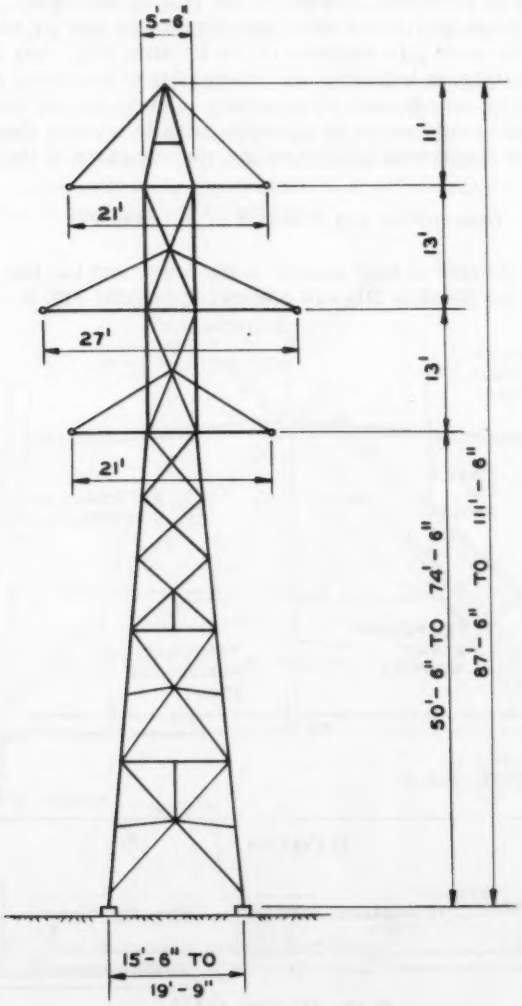


TYPICAL LINE TOWER
FIG. 1

The four faces of the towers are similar and are made of structural steel angles for the main tower legs, horizontal struts, and diagonal compression or tension members.

The foundation consists of stub angles attached to steel angle grillage buried directly in the ground or of stub angles encased in concrete foundations. In all cases the stub angle projects above grade sufficiently for splicing to the tower leg angles.

The maximum weight of tower, conductors, and extension steel is estimated to be about twelve tons.



TYPICAL ANCHOR TOWER
FIG. 2

Preparation of Tower for Raising

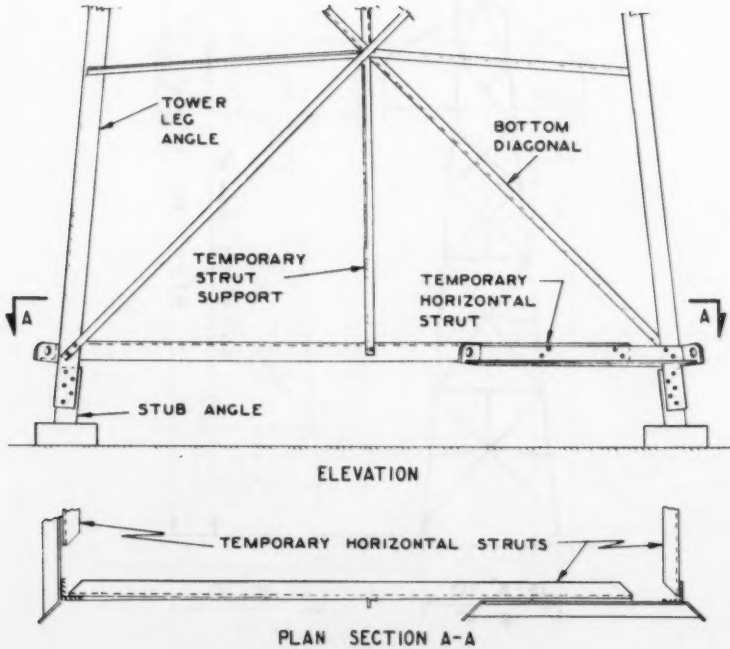
Temporary horizontal struts are attached to the tower using the existing bolt holes at the connection of the lower end of the bottom diagonal to the tower leg angle; see Fig. 3. These temporary struts are adjustable to suit the various base dimensions of the towers. One end of each temporary strut has a two and one-half inch diameter hole to engage the hook of the Hoisting Rig snatch block. This provides a lifting point at each corner of the tower.

A roller guide outrigger, Fig. 4, is then attached to the temporary horizontal struts at each corner. These outriggers extend out and away from the tower at an angle of forty-five degrees to the face of tne tower.

These roller guide outriggers were designed to act merely as a means of gently engaging the main pipe columns of the Hoisting Rig. Any deflection or distortion of an outrigger indicates that the raising of the tower is not proceeding in a smooth, steady, and level manner. Excessive deflection or distortion at one guide would cause an opposite guide to become disengaged and would result in an unbalanced loading on the various parts of the Hoisting Rig.

Description and Erection of Hoisting Rig

The ground is leveled at each corner of the tower and the four bases for the main columns of the Hoisting Rig are placed in position, Fig. 5. The base

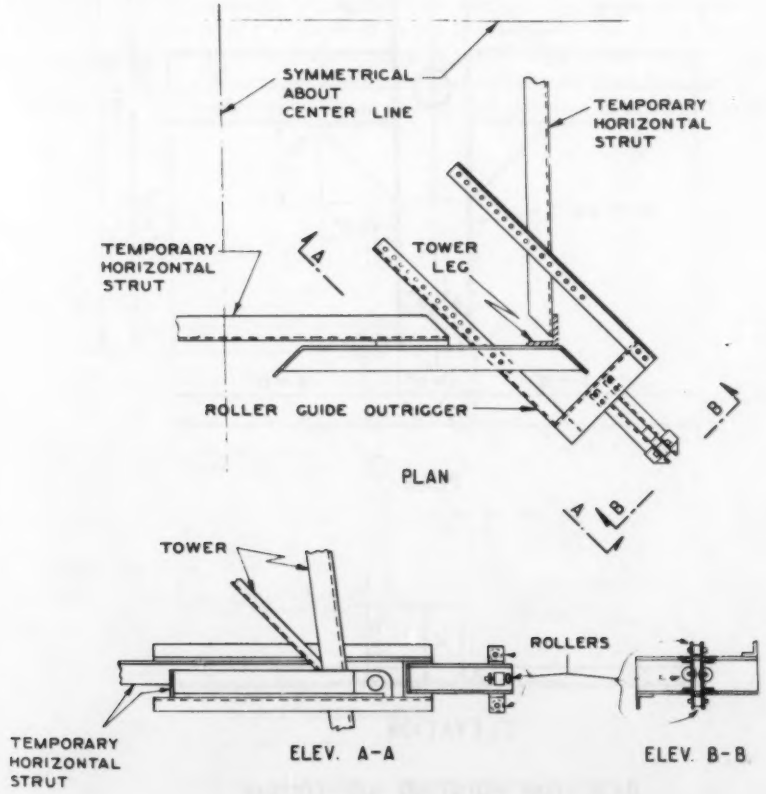


TEMPORARY STRUTS ON TOWER
FIG.3

consists of 2 - 12 inch steel channels welded together to form a cross. The flanges of the channels toe up to produce a flat, smooth bearing surface. At the intersection of these two channels, a twelve inch length of five inch diameter pipe was welded to stand in a vertical position. The bases were designed to keep soil bearing pressures below 1,000 pounds per square foot.

Twenty foot high, six-inch pipe columns are then placed in position over the five-inch pipes on the bases. The columns are turned to such a position that will cause a continuous quarter inch plate, which is welded to the pipe columns, to engage the roller end of the roller guide outriggers previously attached to the tower. The columns are then tied together, at the top and bottom, by four-inch pipe struts. These struts are spliced at their center in such a manner as to make them adjustable to fit any spacing of columns; see Fig. 6.

There is no internal bracing between columns and struts, but the tops of the columns are held in place by two anchor guys at each column. The guys are three-eighths inch wire rope and the anchors are the screw type earth anchors with a three-quarter inch diameter rod. The guys extend down from the tops of the columns at an angle of about forty-five degrees in a vertical plane through the center line of the columns.

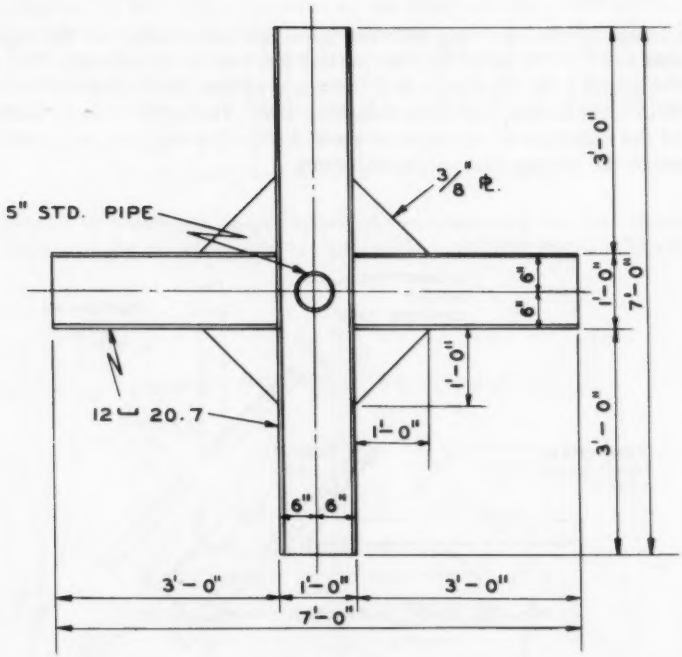


ROLLER GUIDE OUTRIGGER

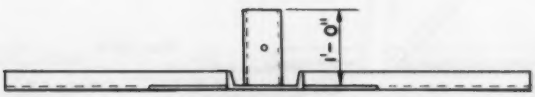
FIG. 4

Ten-inch wire rope sheaves are mounted on the tops of the columns. The sheaves are mounted vertically on a one-half inch horizontal plate which is welded to one end of a twelve-inch length of five-inch pipe. The five-inch pipe is inserted into the six-inch pipe columns, thus the sheave assembly has freedom to rotate.

Snatch Blocks, Fig. 7, each with a ten-inch sheave and swivel hook, are then installed, one at the end of each temporary horizontal strut that was previously attached to the tower and one to each pipe column near the base. Five-eighths-inch wire rope is then fastened near the top of each pipe column and threaded through the snatch blocks at the ends of the horizontal struts, over



PLAN



ELEVATION

BASE FOR HOISTING RIG COLUMN
FIG. 5

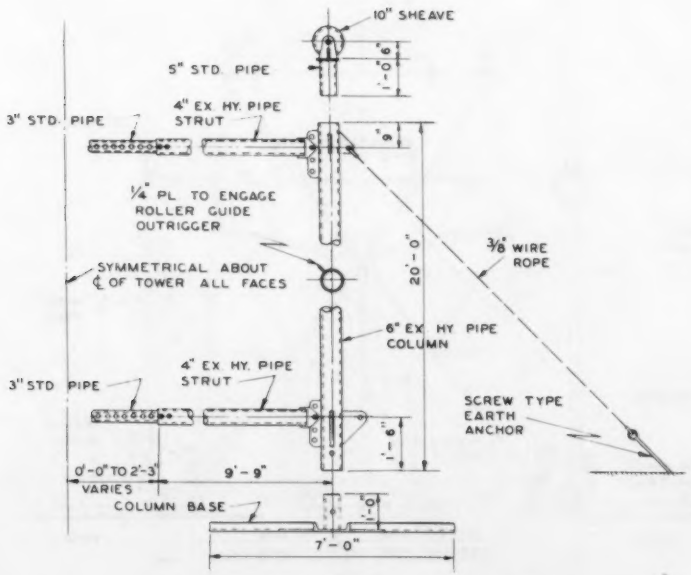
the sheaves at the top of the pipe columns and down through the snatch blocks near the bottom of the pipe columns.

The five-eighths-inch wire ropes from the two pipe columns on one side of the tower are connected together to form a yoke; Fig. 8. A third wire rope from a winch mounted on the front end of a standard line truck is connected to the yoke by means of a snatch block to permit equalizing of line tensions. The wire ropes from the other two pipe columns are similarly attached to the winch mounted on another line truck. These line trucks are generally located on opposite sides of the tower and on the center line of the transmission line.

Manila rope lines, Fig. 9, are attached to the two faces of the tower that are parallel to the Transmission Line at a point just below the bottom cross-arm. These lines extend down at an angle of about forty-five degrees to grade where they are gently snubbed on some convenient anchorage. Their purpose is to control tilting of the tower in a direction at right angles to the transmission line. The conductors themselves control tilting in the direction of the transmission line.

Operation of Hoisting Rig and Installation of Tower Extension

The wire rope lines to the truck mounted winches are now made taut and the weight of the tower is transferred to the hoisting rig and winch lines by disconnecting the bolts at the connections of tower legs to the anchor stub angles. The winches then raise the tower about three or four feet above grade; Fig. 10. While in this position new splice plates are attached to the



HOISTING RIG COLUMN WITH PIPE STRUTS & GUYS
FIG. 6

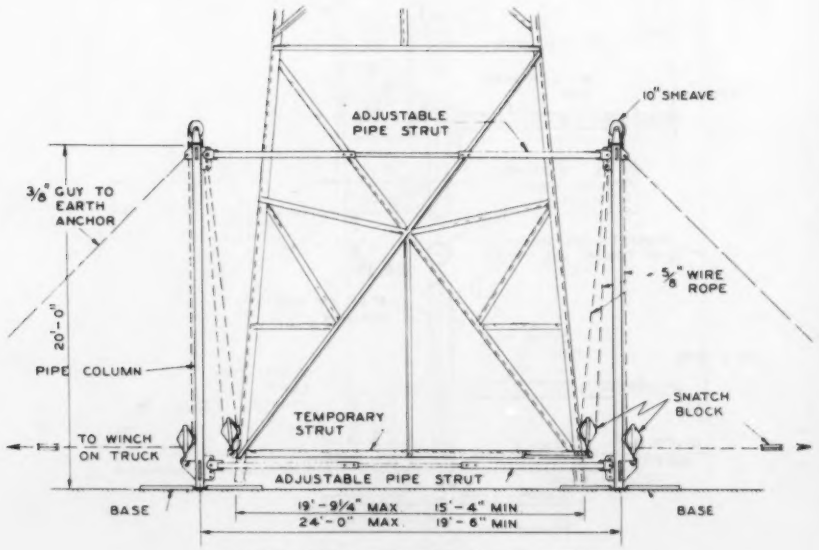
bottom of the legs of the tower. Horizontal struts and diagonal bracing are attached to these splice plates to hold the tower square at its base. The extensions to the tower legs and other diagonal bracing are hung by one bolt to the new splice plates and horizontal struts so that they will fall into position while the tower is raised.

The truck-mounted winches then continue to raise the tower. The manila rope lines attached to the tower near the bottom crossarm are slowly slacked off as the tower is raised but continue to maintain a steadying influence on the tower. The roller guides are watched constantly to see that none of them become disengaged. As the tower rises the new leg angles of the extension swing into a vertical position and when the splice plates at the bottom of the leg angles of the extension meet the stub angles of the anchorage they are bolted in place. The bolting of the top of the leg angles of the extension to the tower leg angles is then completed and the remaining bracing members of the extension are installed and bolted.

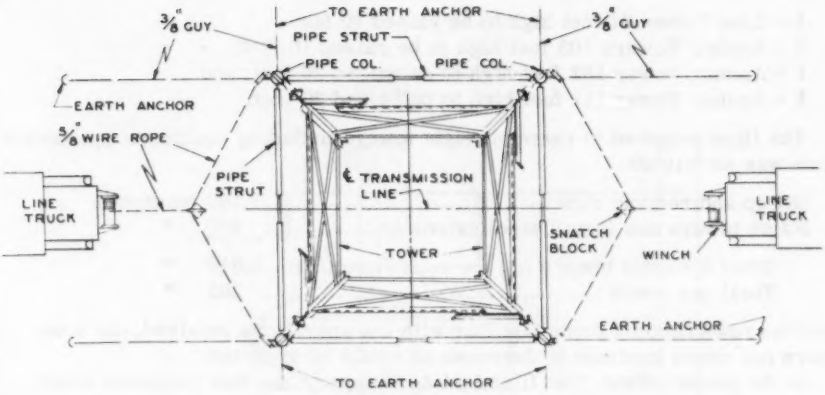
The weight of the tower is now transferred to the tower extension by slacking all lines from the winches, and the hoisting rig, temporary horizontal struts and roller guide outriggers are ready for removal.

This is a description of the method of raising a tower and installing a ten-foot extension. Fig. 11 shows a tower with a ten-foot extension completed. Additional ten-foot sections can be added to the hoisting rig and when securely guyed, any height of lift can be made.

All material used in this tower raising operation was purchased or fabricated locally and is completely re-usable. It is stored in a Company warehouse and is delivered to the job site by truck as required.

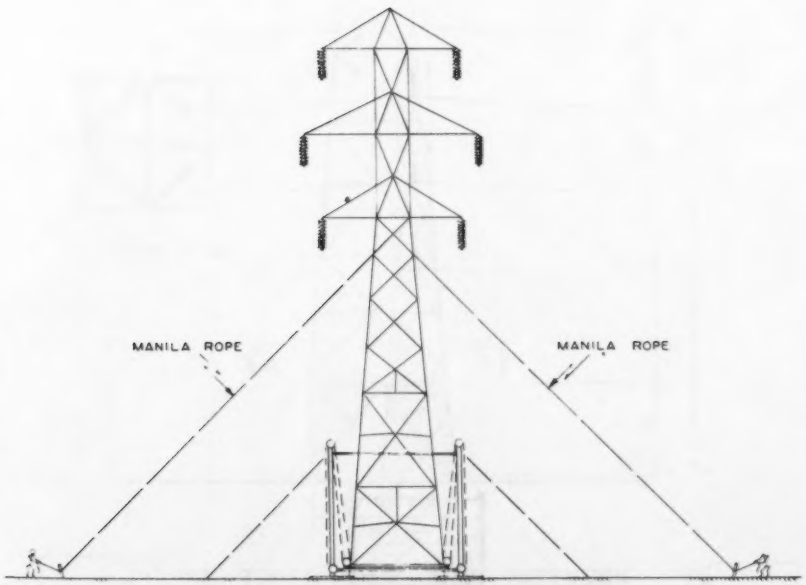


SNATCH BLOCKS AND WIRE ROPE IN PLACE
FIG. 7



PLAN OF WIRE ROPE LINES FROM TOWER TO WINCH

FIG 8



LINES NEAR BOTTOM CROSSARM

FIG 9

The Illuminating Company recently raised a series of eight towers, consisting of

1 - Line Tower 87 feet high to be raised 10 feet
5 - Anchor Towers 103 feet high to be raised 10 feet
1 - Anchor Tower 103 feet high to be raised 20 feet, and
1 - Anchor Tower 111 feet high to be raised 20 feet.

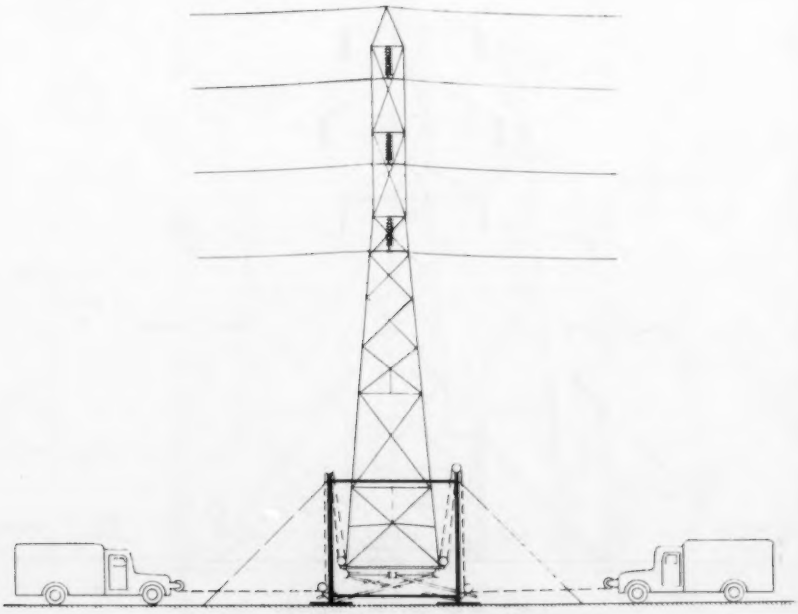
The time required to raise all eight towers including equipment operator's time was as follows:

Set up and remove rigs	1,180	manhours
Raise towers and install steel extension	460	"
Total for eight towers	1,640	"
Total per tower	205	"

As the crews become more familiar with the operations involved, the manhours per tower continue to decrease as would be expected.

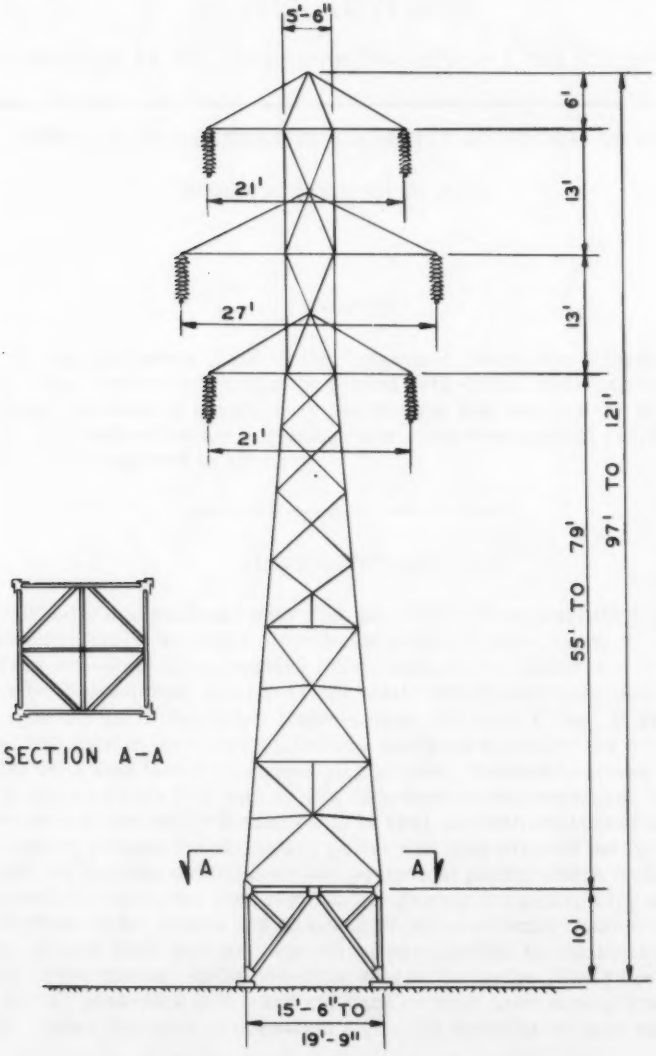
At the present time, The Illuminating Company has four complete tower hoisting rigs. Crews are able to prepare four towers for raising and raise them in as short a time as possible in case the conductors must be de-energized.

The operation was found to be very successful in that it permits the use of standard line trucks and small boom-cranes instead of renting mobile-cranes.



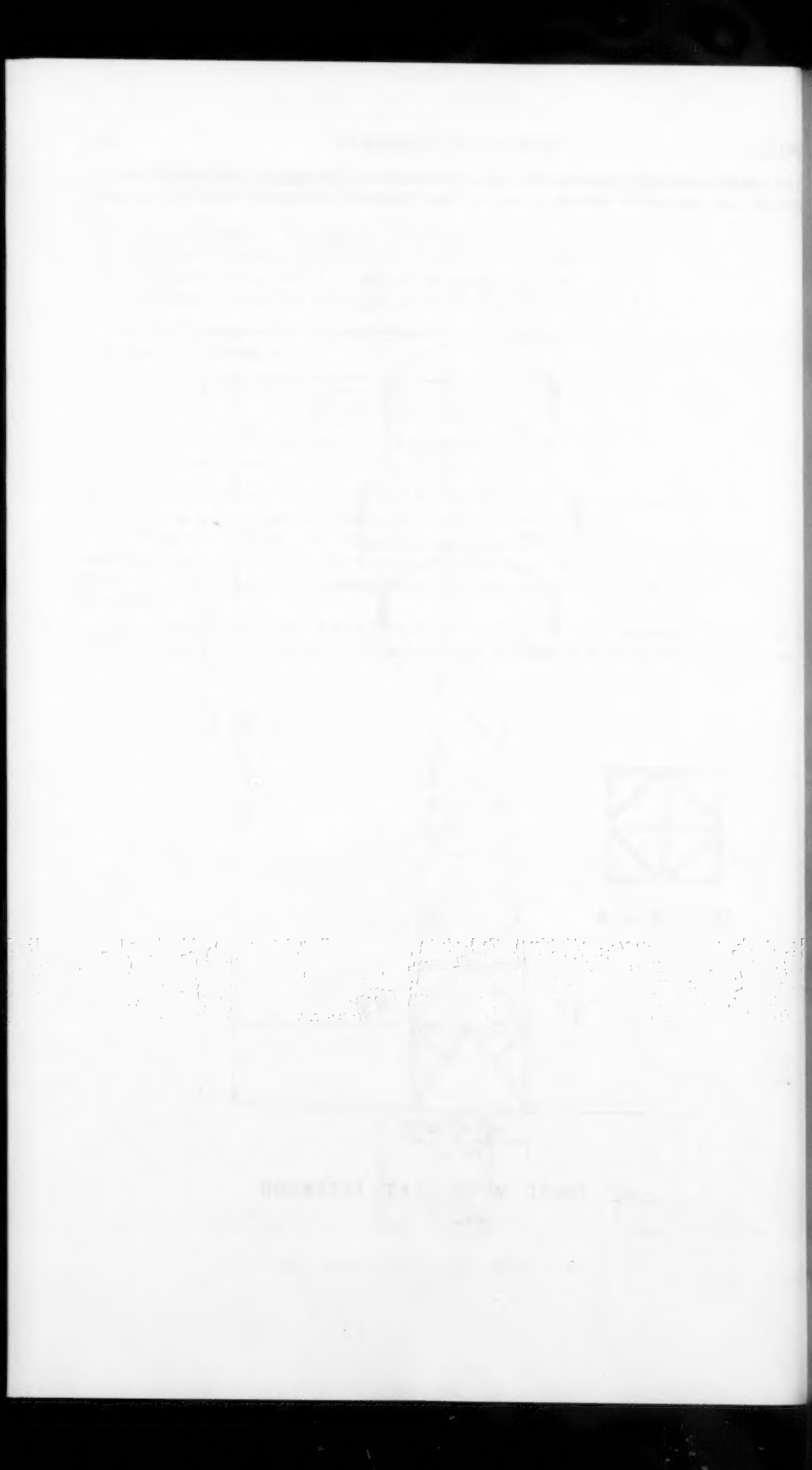
TOWER RAISED ABOUT 4 FEET ABOVE GRADE
FIG. 10

The conductors may or may not remain energized throughout the entire procedure and the work therefore can be done without critical scheduling.



LINE TOWER WITH 10 FT. EXTENSION

FIG. 11



Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ADDITION TO GENERATING CAPACITY AT WILSON DAM[a]

Robert A. Monroe,[1] F. ASCE

SYNOPSIS

Wilson Dam and power plant on the Tennessee River near Florence, Alabama, was constructed during the period 1918-1925. This paper discusses the principal features of the planning and design, and outlines the major problems of construction for a 3-unit power plant extension of 162,000-kw capacity to be completed in 1961.

INTRODUCTION

As originally designed and operated, the TVA power generation system was predominantly hydro with only a limited amount of steam support. By 1950 most of the economically attractive hydro sites on the Tennessee River watershed had been developed, and the empty stalls which were provided in the initial construction of the hydro stations have now been filled. In general the installed capacity in the hydro plants was designed to utilize the normal regulated flow on a load factor of around 70 per cent. Secondary power generated during periods of high flow was sold to industry at reduced rates. With the completion of Johnsonville Steam Plant in 1951 the introduction of adequate steam support primed the secondary power and also allowed the hydro plants to be used for peaking during low-flow periods at plants where sufficient pondage was available for flow regulation. During periods of high stream flow the main river hydro plants and on occasion some tributary plants operate to capacity on base load, and peak variations are handled by the steam plants and tributary hydro plants. In the low-flow season the hydro plants are heavily loaded during peak-load hours and cut back or shut down during low-load periods. Under this plan of operation a greater installation than was originally

Note: Discussion open until March 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2209 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 5, October, 1959.

a. Presented at the May 1959 ASCE Convention in Cleveland, Ohio.

1. Chf. Design Engr., TVA, Knoxville, Tenn.

planned may be economically justified at plants where storage is available to regulate the flow for daily peak operation.

The size of the TVA power system has now reached such magnitude that relatively large additions of either energy or capacity can be readily absorbed by adjusting steam generation. The electric load of the TVA service area is currently increasing around 750,000 kw annually. A winter peak of 9,700,000 kw was recorded in January 1959. The total system load for 1958 was in excess of 61 billion kwh equivalent to nearly 7 million continuous kw. The installed capacity by the summer of 1959 when several generating units now under construction will be in service will be around 11,400,000 kw, of which some 7,600,000 kw or about two-thirds is steam and about 3,800,000 kw is hydro.

Planning Considerations

A study of the TVA hydro plants indicates that the Wilson and Wheeler projects on the main river above Florence, Alabama, offer attractive possibilities for installation of additional generating capacity. An over-all view of the Wilson project is shown by Fig. 1. Wilson has the highest head of the main

WILSON PROJECT - GENERAL VIEW

FIGURE 1

Tennessee River plants and therefore suffers less percentage of reduction in head from backwater during the high flows than do most of these plants. See Fig. 2. The Wheeler plant 15 miles upstream from Wilson also loses little head during high flows because the Wilson Dam crest gates hold the fluctuation of the Wheeler tailrace to within a few feet. Wheeler Reservoir affords ample storage for daily regulation of peak flows through both plants. Space is available at both Wilson and Wheeler for powerhouse extensions without making major changes in existing project features. Pickwick Reservoir below Wilson affords adequate tailwater regulation to smooth out peak-load flow variations. The plants are centrally located in the TVA system and have excellent transmission interconnections. During the four winter months—December to March—the flow of the river as regulated by upstream storage is often substantially in excess of present plant capacity. The proposed additional development is financially attractive as compared to the cost of additional steam plant capacity.

The drainage area above Wilson Dam is 30,750 square miles. The average flow for the 62-year period, 1895 to 1956, was 50,600 cubic feet per second. Fig. 3 shows the duration of natural flow and the flow as regulated by present storage for the typical 12-year period, 1921-32 inclusive. The present plant has a dependable peak capacity of 436,000 kw which requires a flow of from 66,000 to 69,000 cfs depending on the head. The average annual output for the period 1951-58 with all 18 units in operation and virtually all present upstream storage reservoirs in use was 2153 million kwh equivalent to 246,000 continuous kw. The annual average plant load factor was 56 per cent. The regulated flow during the eight low-flow months (April to November inclusive) averaged around 35,000 second feet, with a plant load factor of about 50 per cent. This was quite satisfactory for the TVA system as of 1950, but under present conditions a lower load factor can advantageously be utilized.

The volume of Wilson Reservoir is relatively small but that of Wheeler is ample for peaking regulation. Since the flow travel between the two dams is only two hours, the plants are ordinarily operated in tandem. If generating capacity is increased at Wilson, a corresponding increase in flow capacity will be required at Wheeler plant 15 miles upstream because both plants must be operated in parallel to utilize the Wheeler storage for peaking.

Because of the advantageous situation as outlined in the foregoing, decision has been reached to install additional units at both Wilson and Wheeler. The Wilson addition is currently being designed and construction will begin this spring (1959). The Wheeler addition is still in the planning and preliminary design stage and construction is scheduled to commence next year. The main features of the Wilson plant only will be discussed in this paper.

Three types of prime movers, namely, Francis, propeller, and Kaplan turbines were considered for the new Wilson installation, since the usual range of operating head, 78 to 92 feet, is within the normal range for all three types. Installations ranging in capacity from four 25,200-kw generators similar to the present Wilson units to two 160,000-kw generators were studied. Comparative estimates indicated that an installation of three 54,000-kw generators with fixed-blade propeller turbines would provide the highest benefit cost ratio at Wilson. Contracts have been awarded to S. Morgan Smith Company (now owned by Allis-Chalmers) for the turbines, and the Brown Boveri Company of Switzerland for the generators. The manufacturer's expected performance curves for the turbines is shown by Fig. 4. The average annual

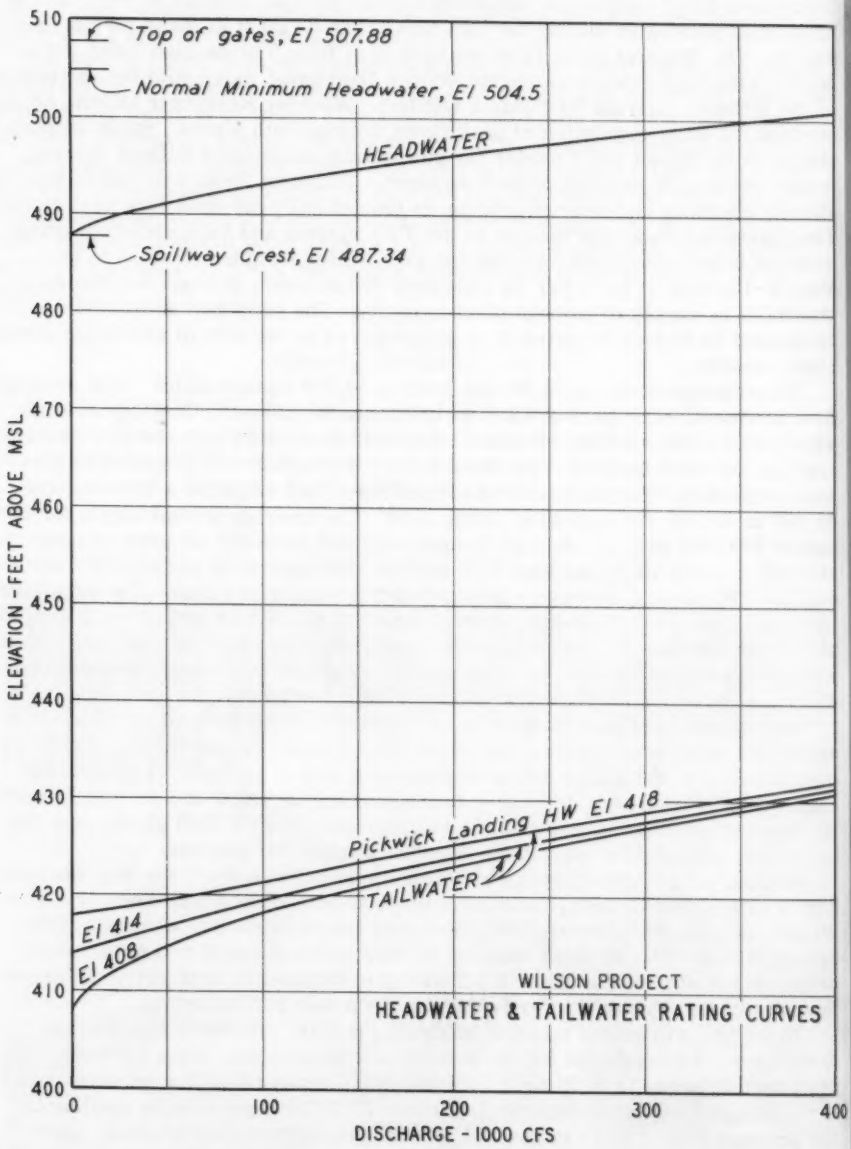

Top of gates, El 507.88
Normal Minimum Headwater, El 504.5
HEADWATER
Spillway Crest, El 487.34
Pickwick Landing HW El 418
TAILWATER
El 414
El 408
WILSON PROJECT
HEADWATER & TAILWATER RATING CURVES
ELEVATION - FEET ABOVE MSL
DISCHARGE - 1000 CFS
510
500
490
480
470
460
450
440
430
420
410
400
0
100
200
300
400

FIGURE 2

increase in generation for the new Wilson units, based on the flow during the typical 12-year period 1921-1932, is estimated to be 162 million kwh annually as shown by Fig. 5.

Design Considerations

Construction of Wilson Dam project was authorized in 1916 and completed in 1925. Hugh L. Cooper designed the project and supervised the construction which was handled by the U. S. Army Engineers by force account. The present installation consists of 18 units aggregating 436,000 kw. A general plan of the present plant is shown by Fig. 6. The prime movers are Francis turbines with Moody draft tubes as shown by Fig. 7 which is a cross section through Unit 18. Eight units had been installed in the plant when TVA took over in 1933, and the remaining 10 were installed by TVA between 1942 and 1950.

The new Units 19 through 21 will be installed in the sluiceway section located between the existing powerhouse and the spillway. Fig. 8 is an architectural rendering of an elevation of the new plant superimposed on a photograph. The sluiceway section is a nonoverflow concrete dam consisting of five blocks separated by contraction joints. Three blocks are 46 feet wide, one block 50 feet wide, and one 42 feet wide, a total of 230 feet. Piers 4 feet thick at each end of each block support the roadway arches. The sluiceways have not been in use for many years and have been plugged at the upstream end, and the valves and operating machinery have been removed.

The existing powerhouse is of the indoor type and the original idea was to extend this to house the new units. This scheme would have the advantage that

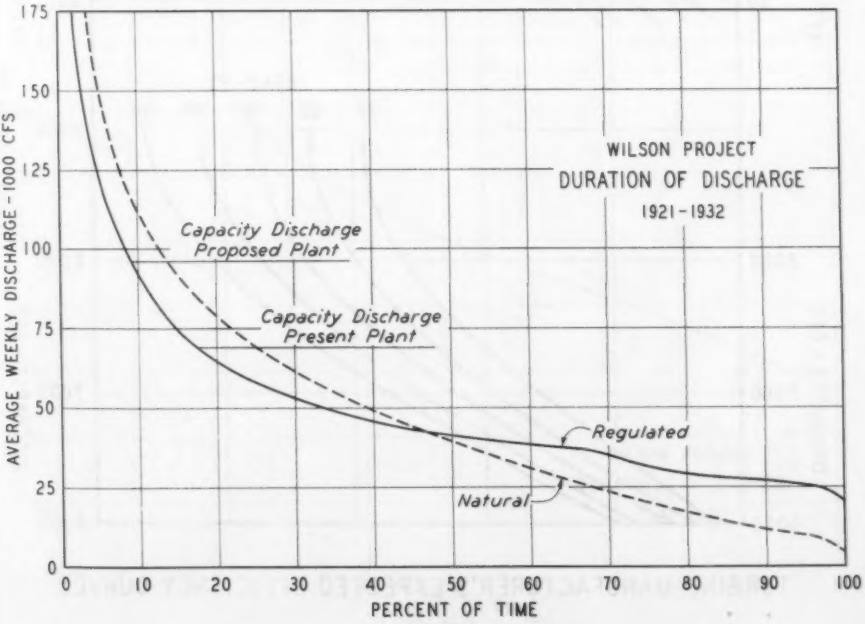
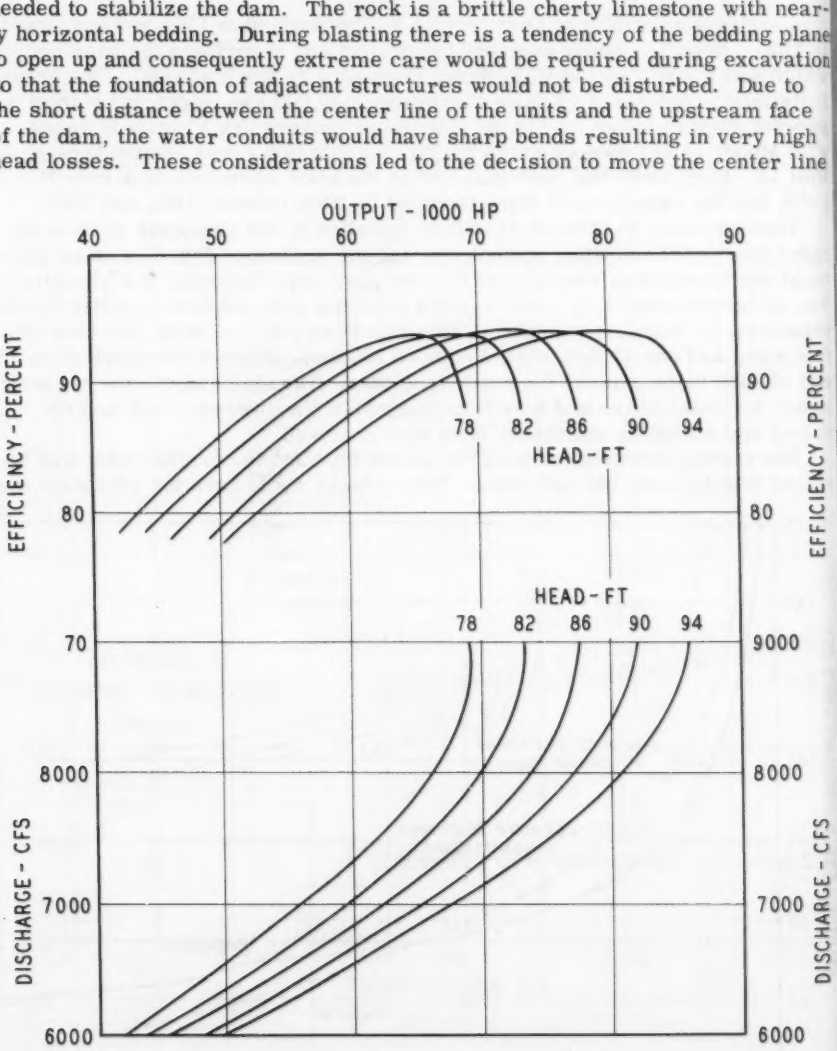


FIGURE 3

existing cranes and erection areas would serve the new units. However, this layout had several undesirable features. The center line of the new units could not be located more than 15 feet downstream of the existing units and consequently the downstream one-third of the sluiceway dam section would have to be cut out to accommodate the new installation. This would result in an unstable structure and prestressed cables or some other means would be needed to stabilize the dam. The rock is a brittle cherty limestone with nearly horizontal bedding. During blasting there is a tendency of the bedding plane to open up and consequently extreme care would be required during excavation so that the foundation of adjacent structures would not be disturbed. Due to the short distance between the center line of the units and the upstream face of the dam, the water conduits would have sharp bends resulting in very high head losses. These considerations led to the decision to move the center line



TURBINE MANUFACTURER'S EXPECTED EFFICIENCY CURVES

FIGURE 4

of the new units further downstream and away from the dam. Heavy equipment like turbine runners and shafts, generator parts, and transformers will have to come through the old powerhouse. This and the additional cost of moving the units further downstream put a practical limit on the offset between the center line of the old and new units. This offset was therefore made 64 feet 6 inches to meet the above requirements.

The outdoor type powerhouse was chosen for these units because in the experience of TVA the outdoor type is more economical than the indoor type for an installation such as this. Fig. 9 is a sectional plan of the new units, and a cross section is shown by Fig. 10. The powerhouse will be remotely controlled from the existing control room, and consequently no employees' facilities are needed. Machine shops, tool rooms, and heavy storage areas are available elsewhere. Compressed air for service and generator brakes and for the governors are available in the old powerhouse and these systems will be

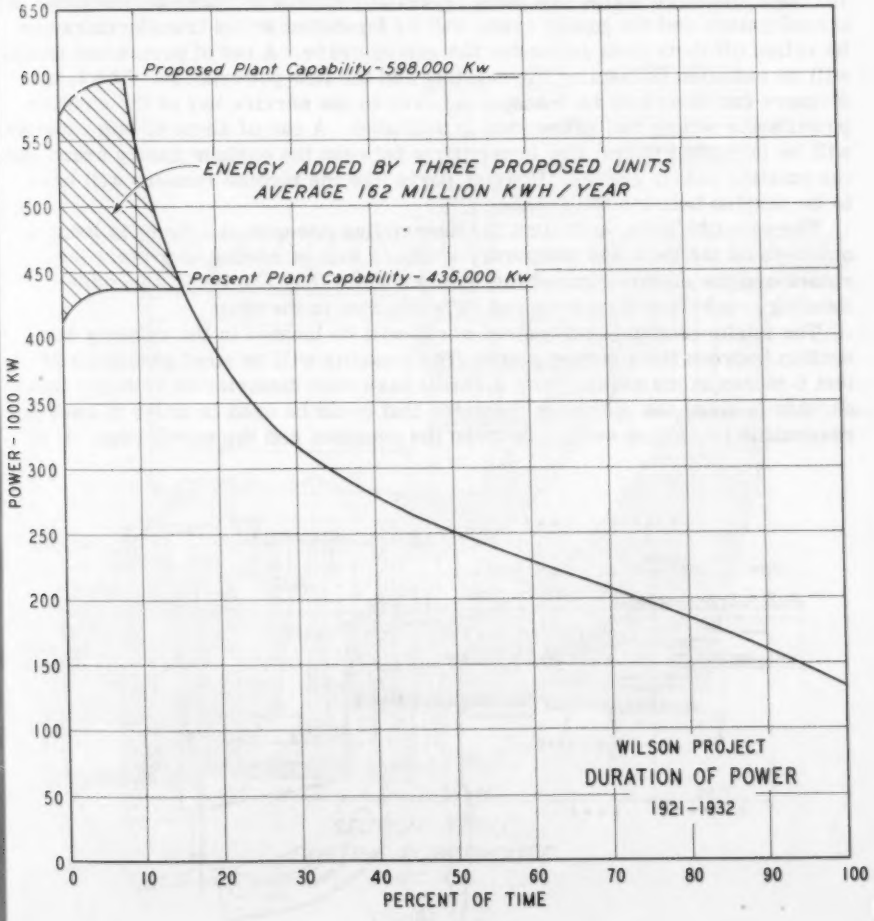


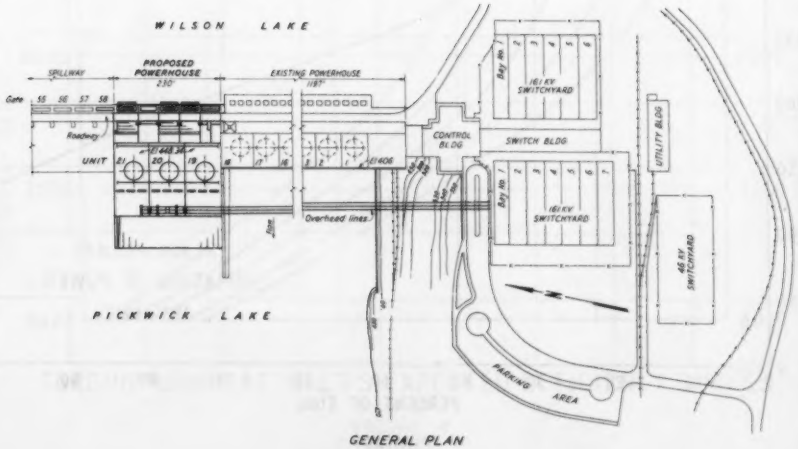
FIGURE 5

extended to serve the new units. Station power service transformers in the old powerhouse also have sufficient capacity to serve the new installation. Consequently only a minimum of facilities and equipment will have to be provided in the new addition. Some of these features are: switchgear, auxiliary boards, lubrication and insulating oil systems, fire-protection system for the transformers, draft tube evacuation system, unwatering, and station drainage system. One intake gate and one set of draft tube gates will be furnished for unwatering one unit at a time. A gantry crane of 300-ton capacity will serve the new units and will also handle the main transformers and the draft tube gates.

The distance between the new installation and the existing transformer yard is about 1,500 feet, and the cost of low tension leads would be prohibitive. The main transformers will therefore be located on the draft tube deck and a river crossing with a span of about 1,400 feet will connect the transformers with the 161-kv switchyard on the left bank. A set of removable tracks between the transformers and the gantry crane will be furnished so the transformers can be rolled off their pads and under the gantry crane. A set of permanent tracks will be installed connecting the existing and the new powerhouse. The transformers can therefore be transported over to the service bay of the existing powerhouse where rail connection is available. A car of about 40-ton capacity will be furnished to transfer heavy parts between the outdoor gantry crane and the existing indoor cranes. Heavier parts like the turbine runners will have to be skidded between the cranes.

The erection area, sufficient for assembling one unit at a time, is located outdoors on the deck and temporary shelters will be needed to protect the rotors and the stators against the weather while they are being assembled. Bearing brackets and runners can be worked on in the open.

The intake conduits and control works will be located in the existing dam section between the roadway piers. The conduits will be steel penstocks 28 feet 6 inches in diameter. With a scroll case inlet diameter of 27 feet 6 inches, this is about the minimum diameter that could be used in order to have a reasonable transition section between the penstock and the scroll case. It is



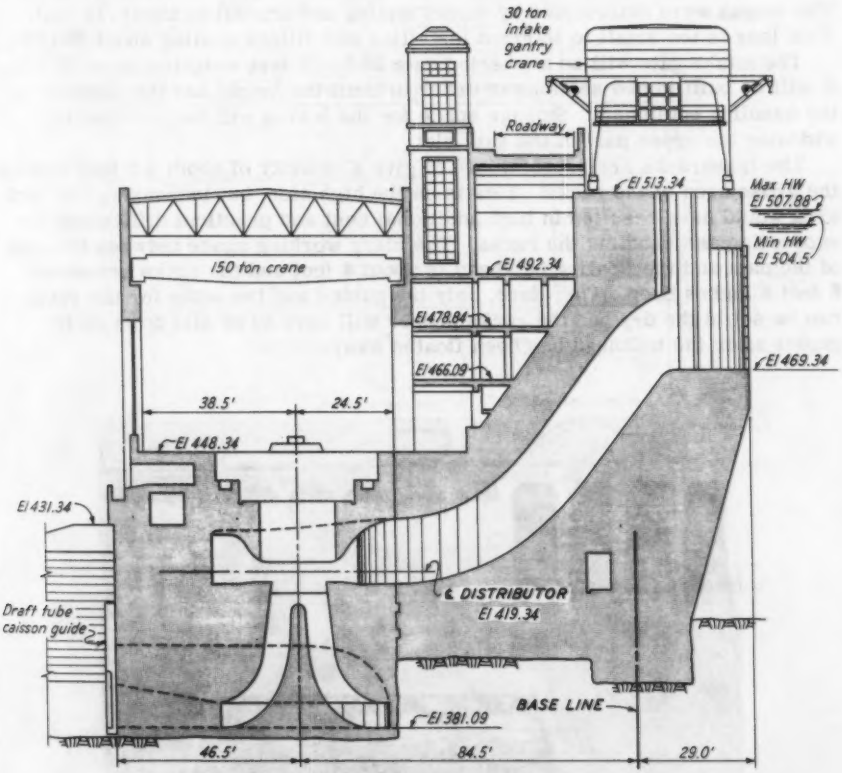
GENERAL PLAN

FIGURE 6

also about the maximum size that could be installed between the roadway piers. The velocity at rated head and discharge is about 13 feet per second which is well in line with TVA practice.

The penstocks are offset vertically and horizontally between the intake face and the scroll case. This is achieved by using two elbows, with a straight tangent piece in between, rotated in "stove pipe" fashion to meet end alignments. The length of the conduits averages about 130 feet. The radii of the bends are 57 feet measured on the center line. The ratio of bend radius to conduit diameter is too low for best flow condition, but is the optimum with the limited space available.

The penstock plate thickness was set at 1 inch. For convenient handling this is about the minimum plate thickness that can be used for this size penstock. Welding will be used throughout, and all welds will be X-rayed to ascertain the soundness of the work. If shipping facilities permit, the fabricator will ship the penstock in 6- to 8-foot-long assembled rings, otherwise the



SECTION THRU
EXISTING POWERHOUSE
UNIT 18

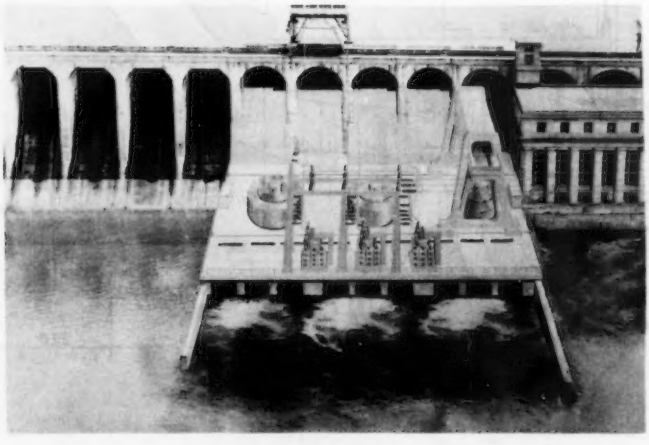
FIGURE 7

shaped plates will be shipped and assembled into rings at the site. The penstocks will have expansion joints midway between the two elbows and likewise between the downstream elbows and the transition sections connecting the penstocks to the scroll cases. The penstocks will be encased in concrete except at the expansion joints and in the area between the unit blocks and the dam section. Upstream of the gate slots percolation rings and bond between the steel and the concrete will form a seal against headwater. The elbow sections will have an elastic coating on the upper two-thirds of the periphery and the straight section will be supported on rockers and have an elastic coating over the entire periphery. In this way a minimum of restraint will be imposed on the penstock. This is essential as the water in the Wilson Lake varies between 85-90 degrees F in the summer and 45 degrees F in the winter, and the penstock must be free to expand and contract with the change in temperature.

The gate slots are rectangular in shape to accommodate the gate. It was felt that the transition from a rectangular gate slot to a circular penstock could result in considerable head loss and that slot fillers could be justified. The losses were determined by model testing and amount to about .15 foot. This loss is too small to warrant installing slot fillers costing about $60,000.

The intake gate will be a wheeled gate 29 by 29 feet weighing about 70 tons. It will be built in two sections in order to limit the height and the capacity of the handling equipment. Storage space for the leaves will be provided by widening the upper part of the gate slot.

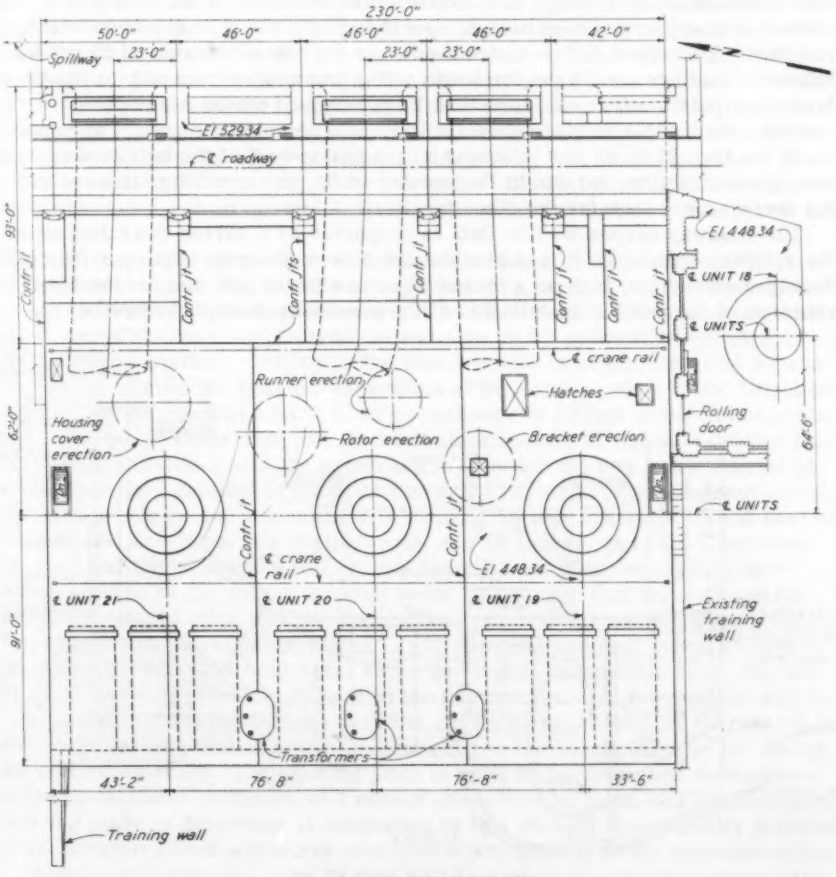
The trashracks are proportioned to give a velocity of about 4.5 feet through the gross area of the racks. This is on the high side, but increasing the rack area would have resulted in high additional cost and practical difficulties in supporting and handling the racks. The clear working space between the face of the dam and the floating bulkhead is about 4 feet and the racks are about 6 feet 6 inches deep. Therefore, only the guides and the seats for the racks can be set in the dry and the racks proper will have to be slid down on the guides after the bulkhead has been floated away.



DOWNSTREAM ELEVATION

FIGURE 8

Cutting through the existing dam to install the penstocks is a difficult operation. It could have been very much simplified by cofferdamming, but the water is 100 feet deep in this area and a cofferdam closing off the 230-foot section would have cost about $1,200,000. It was therefore decided to use a floating bulkhead, set and sealed against the dam to close off one intake bay at a time. After the intake for Unit 19 has been finished, temporary stoplogs will be inserted in the gate slot, and the bulkhead moved to close off the intake bay for Unit 20. The intake gate will be in place in Unit 20 before the bulkhead is moved to Unit 21. The bulkhead was originally built for construction of the intake culverts for the new Wilson lock, now just being completed across the river. Slight modifications will make the bulkhead also usable for constructing the intake of these units. The bulkhead is of steel with compartments that can be flooded or unwatered. It can be towed in horizontal

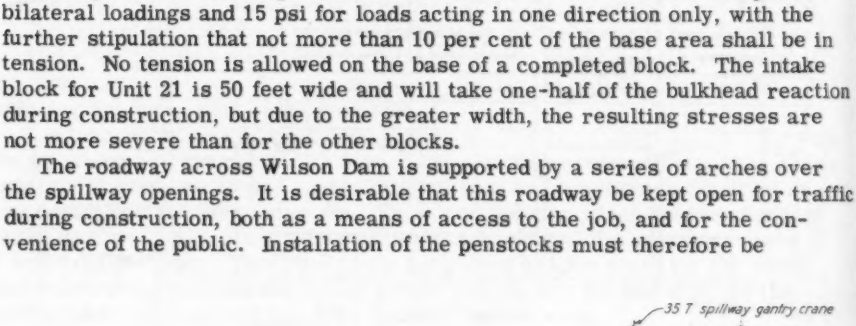


PLAN OF NEW UNITS

FIGURE 9

position, and by flooding the lower compartments raised to a vertical position, pulled tight against the dam and the area between the bulkhead and the dam unwatered. It is about 75 feet high with a clear span of 46 feet between supports. The reaction is therefore taken by the blocks adjacent to the one under construction. This is essential because during one stage of construction about half of the mass is removed and the remainder of the block is barely stable even without the reaction from the bulkhead. The contraction joints between the blocks have an asphalt seal 8 feet downstream of the face of the dam. The contraction joints falling within the unwatered bulkhead will be dry and it must be assumed full water pressure is acting upstream of the seal in all joints outside of the bulkhead. Therefore the block adjacent to the one under construction will have to carry the reaction from the bulkhead as well as the lateral load from the water in the contraction joints. These loads, in addition to the normal load, tax the stability of these blocks to the very limit. Since this combination of loadings is a construction condition, it was felt that a deviation from normal dam design, specifying no tension on the base, was justified. Therefore design instructions allow a corner tension of 25 psi for bilateral loadings and 15 psi for loads acting in one direction only, with the further stipulation that not more than 10 per cent of the base area shall be in tension. No tension is allowed on the base of a completed block. The intake block for Unit 21 is 50 feet wide and will take one-half of the bulkhead reaction during construction, but due to the greater width, the resulting stresses are not more severe than for the other blocks.

The roadway across Wilson Dam is supported by a series of arches over the spillway openings. It is desirable that this roadway be kept open for traffic during construction, both as a means of access to the job, and for the convenience of the public. Installation of the penstocks must therefore be



CROSS SECTION THROUGH NEW UNIT

FIGURE 10

accomplished without causing damage to the roadway piers and arches. Trenching for the penstocks will leave the 4-foot piers free standing from the springing line of the arches to the bottom of the trenches, a distance of 60 feet. These structures are not reinforced and unbalanced lateral loads from the arches could cause the piers to crack and possibly collapse. It is expected that the contraction joints between the arches are frozen at the springing line so that the side thrusts of the arches will balance each other; measuring devices have been installed to verify this. Should the measurements show that the joints open and close with the variation in temperature, means must be found to take the horizontal component of the arch thrust.

The unit blocks are separated from each other and from the intake by contraction joints. Each block is 76 feet 8 inches wide and 147 feet long. The scroll case will be welded and pressure-tested for leakage. It has an inlet diameter of 27 feet 6 inches and the over-all width was limited to 70 feet to give a minimum concrete cover of 3 feet 4 inches. An elastic coating will be applied to the upper two-thirds of the scroll case periphery to prevent load transfer from the case to the surrounding concrete. To prevent distortion of the case and other embedded parts, the rate of placing concrete will be limited to 1 foot per hour with a stop every 2 feet until the concrete has taken its initial set. Otherwise the unit blocks will be placed in 5-foot lifts with the time interval between lifts varying with the placing and air temperature in order to control the cracking of the concrete. The turbine blocks are of conventional layout and no special difficulties in either design or construction are anticipated.

A cellular cofferdam will be constructed to unwater the construction area of the powerhouse. For reasons of economy it is carried to an elevation expected to be over-topped once a year. It will be connected to a training wall at the end of the existing powerhouse and also to the spillway block adjacent to the sluiceway section. The top of the training wall is at elevation 424.34 and the bottom follows the tailrace excavation of the existing units. The height of the wall varies therefore from 44 at powerhouse to 12 feet at the downstream end. The top is 4 feet wide and the back slope is 4 on 10 and it is evident that the wall is not stable with the construction area for the new units unwatered. It was therefore decided to stabilize the wall by the use of prestressed cables. The excavation for the new units will be about 20 feet below the lowest part of the wall and only a narrow shelf of rock, say 12 inches, can be left between the wall and the excavation for the new installation. Considering the horizontal bedding of the rock it was felt there was danger that the wall and its foundations could slide into the excavation area during blasting operations. The cables will therefore be slanted away from the construction area, proportioned to resist the horizontal shear due to full hydrostatic head, and will be anchored well below the bottom of the new excavation. A causeway will be constructed to Jackson Island where the rock excavation will be spoiled. It is also expected that material will be available on the island suitable for filling the cofferdam cells. The existing piles of rock in the river bed downstream of the powerhouse extension will reduce the operating head, and investigation is being made to determine if excavation of this rock is economically justified. If so, Jackson Island will serve as a convenient spoil area for this excavation.

Heavy parts weighing over 25 tons will have to be transported through the existing powerhouse to the construction area. Lighter parts will be trucked over the Wilson Dam roadway and lowered by crane to the construction area.

Concrete will also come over the roadway, but as yet it has not been decided how to transfer the concrete from the roadway to the working area.

Economic Considerations

The estimated cost of the three 54,000-kw units installed at Wilson Dam is $24,000,000 or $148 per kw which includes the cost of step-up transformers and switching station. The size of units for the Wheeler addition has not been finally determined; but the cost per kw, which will be higher than Wilson because of the lower head, has been preliminarily estimated to be $16,000,000 or about $248 per kw.

The three new Wilson units are estimated to generate 162 million kwh annually, and to add 162,000 kw of peak capacity. That the expenditure is economically justified is indicated by the following cost analysis:

Capital cost of Wilson addition (including switchyard)		$24,000,000
Annual cost		
Interest at 4%	$960,000	
Depreciation 40-year life - 4% S.F.	240,000	
Operation and maintenance (additional to present plant)	80,000	
		1,280,000
Annual energy 162,000,000 kwh @ 2.5 mills		405,000
Remaining cost chargeable to dependable capacity		875,000
Annual cost per kw dependable capacity		5.40

The value assigned by TVA to dependable peak capacity for economic appraisals is in the order of $10 per kw per year. On this criterion the cost of additional peak capacity at Wilson is very reasonable. The Wheeler addition will cost more per kw, but the cost of peak from the combined plants will still be well below its economic value.

ACKNOWLEDGMENTS

The planning, design, and construction of this project is being handled by TVA forces under the general direction of George K. Leonard, Chief Engineer. The planning report was prepared by the Division of Water Control Planning of which Reed A. Elliot is Chief Water Control Planning Engineer. Construction will be under the supervision of Henry T. Lofft, Chief Construction Engineer, with Warren McMahan as Project Manager. Design is being handled by the Division of Design of TVA under the supervision of Robert A. Monroe, Chief Design Engineer. The discussion of design considerations was prepared by O. Lavik, Staff Civil Design Engineer.

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DESIGN, OPERATION, AND MAINTENANCE OF INTAKES, RACKS, AND BOOMS

Progress Report of the Committee on Operation and Maintenance of Hydroelectric Generating Stations of the Power Division

ABSTRACT

This report by the Committee on Operation and Maintenance of Hydroelectric Generating Station compiles certain available data regarding intakes, racks and booms. Although the paper deals primarily with operating and maintenance problems, it presents a reasonably broad discussion of the design philosophy pertaining to intakes, racks and log booms.

INTRODUCTION

This report is intended to compile certain available data regarding intakes, racks and booms. Although the assignments of this Committee deal primarily with Operation and Maintenance Problems, it was found that a reasonably broad discussion of the design philosophy pertaining to intakes, racks and log booms would be an essential part of this discussion.

It is not intended that the theory of the hydraulics of intakes will be covered, but certain general statements will be made regarding approach conditions to intakes insofar as they affect the trash racks. Nor will intake gates, guides or hoists be discussed in detail. Herein, the term boom means a device which, in its simplest form, is frequently constructed of a string of logs fastened together by cables or chains for the purpose of diverting or gathering trash floating on the stream as it approaches the intake.

These three appurtenances of an hydroelectric station are quite closely related and in many cases must be considered together in the design of the plant. The depth of the intake, that is, the entrance to the conduits or penstocks, determines the effective or useable area for the trash racks. In some

Note: Discussion open until March 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2226 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 5, October, 1959.

installations, the depth of the penstock opening is so shallow as to seriously limit the available area for screens. As a result, the approach velocities, as well as the velocities through the racks, may be higher than is normally desired. If the trash boom is to be located relatively near to the intake, these high velocities may affect the conditions for which the boom must be designed.

In the process of collecting data for this paper, one of the first factors of which the authors were reminded, was the effect of geographical location upon the basic design criteria. For example, a plant in northern New York State may have low temperature problems similar to or greater than, a plant in the Pacific Northwest. In addition, the New York plant may be troubled by large quantities of leaves floating on the streams in the fall. In place of such deciduous trash, the Pacific Northwest plants may be plagued by logging debris. A plant in the North central states may be relatively free of drift while a plant in Southern climes may be troubled with heavy floats of water hyacinths. On several plants on the Missouri River, trash has not presented a problem. This is somewhat surprising since only a small amount of clearing, in the immediate vicinity of the dam, was done. For one reason or another, it appears that the drift has a tendency to strand itself along the shore line. This may be the result of the characteristics of the banks.

During the preparation of this report, an interesting feature found in the design of many older plants throughout the country was brought out by the descriptions of some plants built along the Niagara and the Raquette Rivers of New York State between 1896 and 1918. These particular descriptions indicated relatively shallow intakes with sloping racks enclosed in a housing. Hand raking was contemplated in the design. The water velocities through the racks were low, varying from 1.0 to 1.5 fps. The racks were designed to withstand the full hydrostatic head between full forebay elevation and rack bottom elevations. This typical arrangement has been classified as a reflection of the influence of paper mill plant design. Quite similar structures of the same era are found in other parts of the country and, whether the mills from which the design emanated produced paper, flour or other products, the general appearances are the same and quite characteristic of plants constructed close to the turn of the century. This wide divergence in the types of problems affecting plants in different localities requires that the plant designer make careful studies of the local conditions in order to properly develop his design criteria. While this is usually possible through contacts with operators of plants in the vicinity, it is usually quite difficult to establish quantitative evaluations. A review of some of the more recent hydroelectric developments of the U. S. Army Engineer Corps in the Missouri River Basin, for example, indicates that no appreciable difficulty has been experienced from drift and for the most part there is no accumulation of trash. A number of intakes in Northern California are found to have racks sloping at 45° with low rack velocities. Hand raking is found to be facilitated since the debris tends to ride up the rack toward the water surface. Such racks are commonly designed for full hydrostatic head.

Since this paper deals with the maintenance and operation, as well as design, of the features under consideration, it must be kept in mind that the overall economy must be the major criterion. To some it might appear desirable to have all operation handled by push button from the operator's arm chair or, on the other hand, to others it might appear desirable to have much of the operation done by manual labor. If the designer is to satisfy a prudent management, he must provide features for the plant of such design as will

result in the minimum annual charges for that feature. Interest, taxes, amortization, overheads, operating payrolls, maintenance expenses and many other annual charges must be weighed before one design philosophy can be selected as being superior to another.

Intakes

Insofar as hydroelectric power plants are concerned, intakes can be said to come under one of two major types. The first type is that which is found to be integral with the non-overflow section of the dam and the second type is that in which the penstocks are carried in tunnels which have their entrances some distance from the dam.

In the first case it is usually quite simple for the designer to obtain adequate submergence of penstock entrance unless the dam is of extremely low height. In such cases the entrance to the penstock must be relatively large. Velocities through the intake gate are generally acceptable if they are equal to or less than the square root of the head above the center of the intake opening. Should the dam be so low as to make the installation of a conventional type intake with adequate submergence very difficult, a special design may be substituted. One type described herein is the "draft distributor" intake. Intakes having their centerlines only a relatively short distance below the forebay water surface are far more subject to sudden and serious rack clogging, caused either by ice or floating debris, than deeper intakes.

Fig. 1 shows several variations of deep intakes with modifications of entrance conditions including an approach to a straight conduit with head gate and guides on the face of the dam with a relatively small bellmouth entrance, a double bend conduit with gate and bulkhead slot in the dam located a sufficient distance in from the face to avoid losses from the bellmouth, and an inclined intake entrance with a single bend conduit and gate slot and intake within the dam.

In cases where the entrance to the penstock is located at some point other than in the upstream face of the dam, the topography of the area may be such as to require the minimum adequate submergence on account of the relative economies of the situation. This condition is found, for example, where the intake is excavated out of a rock cliff. In this situation every foot of depth becomes extremely expensive in first cost and must be justified by operating saving.

Another situation is frequently encountered at earth fill dams, where a long penstock is used to go through or around the dam. This condition usually gives the designer considerable latitude, insofar as submergence is concerned, but the length of the penstock and the magnitude of the separate intake structure require economic comparisons.

The size of the penstocks selected is, of course, an economic study in itself. We are here concerned only with the economic study pertaining to the intake and trash rack.

In general, the velocity through the net area of the trash rack should not exceed about two feet per second. Under certain conditions, usually encountered in connection with the second or third type of intake as described above, a greater velocity may be found economically justifiable.

It should be noted that the length of the intake cannot be extended indefinitely in order to obtain an adequate area for the trash racks. Conceivably, any

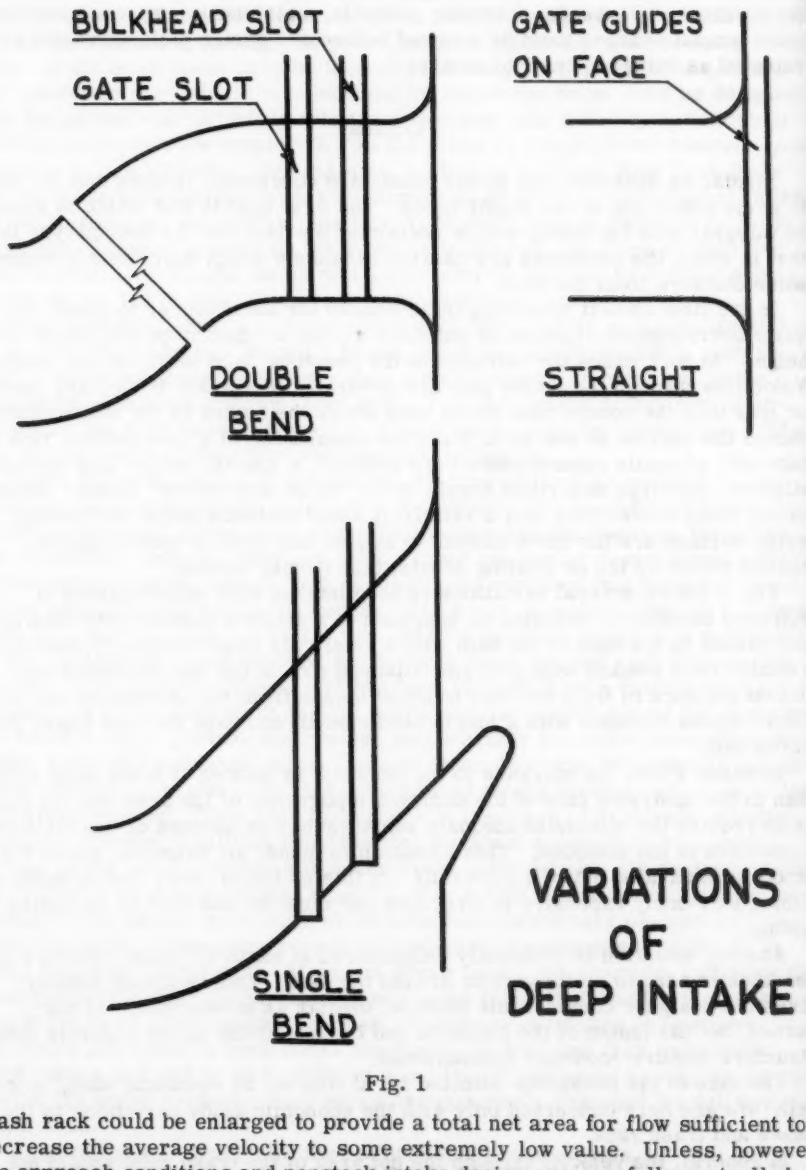


Fig. 1

trash rack could be enlarged to provide a total net area for flow sufficient to decrease the average velocity to some extremely low value. Unless, however, the approach conditions and penstock intake design are carefully controlled, the average velocity will have but little meaning. If we assume a rack structure having this unusually large area located some distance upstream from the actual penstock opening, the velocity of approach pattern in the plane of the racks will assume the shape of an oblate hemispheroid, a prolate hemispheroid, or some similarly curved surface. The maximum velocity through

the racks will be on the axis of the penstock produced. As we move, on the rack surface, away from this point of interception, the velocity will drop in conformity with the shape of the velocity of approach pattern. Since the head loss through the racks will be a function of maximum velocity, it can be seen that there is a definite limitation to the area of the racks, in relation to the area of the penstock opening and the distance between this opening and the racks, which can be considered as effective rack area.

The fact that trash will collect in the area of highest velocity, modified by the bouyancy of the trash, is one of several factors which affect the determination of the distance between the face of the intake proper and the trash racks. Other factors include the required working area above the intake structure, roadway width and similar items. In some designs, we find the racks are actually in the plane of the penstock opening. This is usually the case with relatively low head units located in a powerhouse integral with the dam. In such cases, it is quite probably that, since the entire penstock is converging to provide uniform acceleration of flow, a large part of the rack area can be assumed to be effective area. Where the racks are located upstream from the penstock opening, as discussed previously, this horizontal dimension, for units requiring large penstocks, is frequently equivalent to the diameter of one penstock.

Fig. 2 shows a low velocity intake with the trash racks of adequate area located in the plane of the intake opening with converging flow to the unit.

In general, in the last case cited, it can be said that a length of trash rack equal to three times the diameter (D) of the penstock is about all that can be considered as the effective length of the racks for computing the mean velocity through the racks. Similarly, three times the penstock diameter would be the effective vertical dimension of the racks. Thus it can be seen that, with the racks located one penstock diameter from the intake face, the effective area of trash racks is probably safely expressed by $2.25 \text{ pi } D^2$, or 9 times the penstock area. Thus, if we have penstocks twenty feet in diameter, the effective gross area of trash rack would be about 2800 square feet.

In determining the minimum depth at which the penstock intakes should be located, it is important to consider the submergence which must be provided in order to permit the development of velocity head at the entrance to the penstock. In addition to this value, which is readily calculable, an allowance must be made for the submergence required in order to prevent the development of adversely effective vortices. From a practical standpoint, an empirical allowance of submergence approximately equal to one-third of the diameter of the penstock in addition to the provision for velocity head is usually found to be satisfactory, although this value is frequently set at one-half of the diameter.

In cases of shallow intakes, a special type of intake may be employed. One such type, known as a "draft distributor intake" consists of a submerged conduit extending from the penstock into the forebay with a converging slot at the top or side of the conduit for the entrance of water. This type has been employed where considerable sheet ice and other floating debris may be encountered. It is necessary to maintain the entrance velocity low enough to permit the floating material to continue undisturbed on its natural course and not be sucked down into the collecting conduit. The Chelan Dam in Washington uses such an intake arrangement.

It has been found that shallow intakes, somewhat less than thirty feet, have proven to be acceptable development areas for caddis fly larvae. The

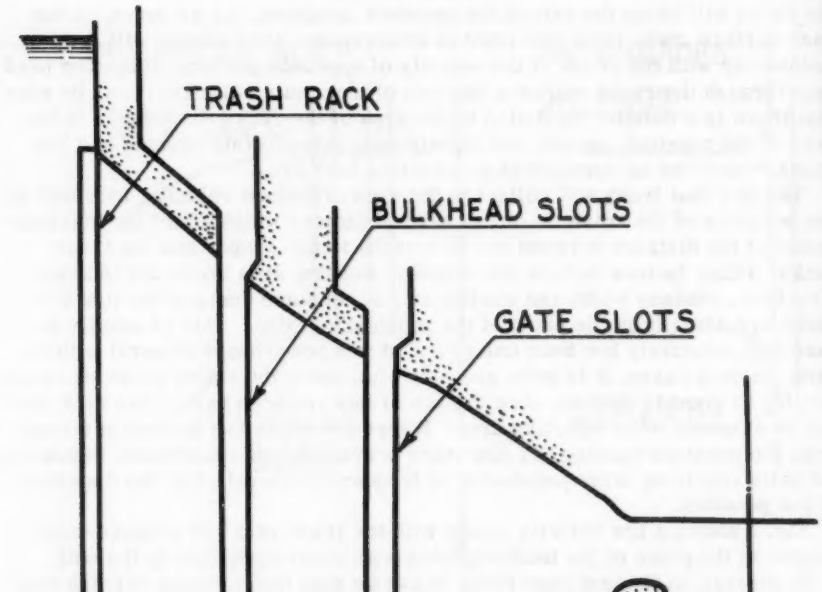


Fig. 2

collection of these flies has, at times, been quite troublesome and has caused appreciable head loss. Similar difficulties have been reported from Japan and elsewhere.

Racks

The necessity for trash racks on hydroelectric plants stems from the possibility of damage to the turbine runner or the wicket gates being caused by relatively large pieces of floating debris. Screens, as commonly used in steam electric stations accomplish, a similar purpose except that the permissible size of debris is much smaller and hence requires horizontal as well as vertical members on the debris restraining face. The fact that a stream

carries very little debris obviously does not eliminate the need for trash racks. It may, as will be seen later, affect the detail design of the racks, the rack cleaning arrangements and the requirements for a trash boom. Streams in lumbering areas, particularly in rugged terrain subject to heavy spring runoffs, carry large amounts of slash and discarded timber. In some instances, notably Dennison Dam on the Red River in Texas, large amounts of drift exist in the reservoir, but, this drift largely beaches itself before reaching the intake. In contradistinction to this, some rivers can be seen to carry the trash in an attenuated line following the thread of the stream and, although seasonal beaching may occur, practically all flotable material finds its way to the intake. It appears that the tendency for drift to accumulate on the shore or to follow the thread of the stream through a reservoir is a function of the velocity of this thread in relation to the velocity pattern of the entire reservoir area.

Trash racks are usually made up of vertical members of steel bars 3/8 to 3/4 inch thick and from 2-1/2 to 5 inches in width and extending vertically for whatever distance is available in order to obtain optimum trash rack area. The spacing of these bars is usually determined by the characteristics of the turbine. Either of two criteria may be the limiting factor in the determination of this spacing. In some instances the size of the maximum timber which can be sheared by the wicket gates may control and in other cases the maximum size of timber which could safely pass through the turbine wheel will control. Ordinarily, values for this bar spacing range from five to nine inches. In some localities, such as Northern California, closer rack spacing has been required by the fish protective organizations. Considerable difficulty is quite obviously caused by clogging of racks when the spacing is made too small. In some special cases, round bars or pipes have been substituted for flat steel bars in the fabrication of the trash racks. The rack structures in the earlier plants were of bolted or riveted construction, but, in recent years this has largely been supplanted by welded construction. This has simplified the structural details and considerably reduced the overall weight of the structures. Some use has been made of wrought iron or stainless steel in trash rack fabrication but at the present time this cannot be said to be common practice.

In situations where there is considerable amount of trash to be expected, a value of five feet of differential head is commonly used in the structural design of the racks. That is, the racks are so designed as to be able to withstand a differential head caused by the clogging of the racks amounting to as much as five feet. In some cases, especially at automatic or remotely controlled plants, it is advisable to design the racks against full hydrostatic head. Since the value of a plant is a direct function of the head, excessive clogging cannot be tolerated unless the height of the intake is only an extremely small part of the head of the units, as in the case of the high head plants of the Sierra Nevada. Obviously, this value will vary considerably depending upon the type and amount of debris which the stream carries. It is also obvious that, without an adequate trash rake, this differential head for design purposes must be much greater, approaching as a maximum the total depth of forebay above the bottom of the racks. Therefore, the provision of a trash rake in the original design will permit economies in trash rack design.

In cold climates, where the temperature of the air frequently drops to below freezing, the tops of steel rack bars should be kept below water surface. Because of the high thermal conductivity of steel, ice forms rapidly on the bars

if they extend into the air, and this anchor ice may rapidly extend to the full depth of the rack structure. This may be accomplished by substituting solid wood panels for the uppermost rack bar panels extending from a point slightly below low water to the top of the trash rack structure.

On the Raquette River in northern New York, five (5) plants develop 100 000 kw through a total head of 470 feet. These plants were built during the period of 1952 to 1957 and incorporate some of the more modern design features and criteria. It is important to note that, in this locality, winter temperatures sometimes drop to -35° or -40° F. The velocity through the net area of the racks is 3.2 fps. Recognition is given to the limitation of effective rack width and height and the width is made the same as the draft tube exit width. Stop-log slots are provided upstream of the trask racks and the stop logs for both intake and draft tubes are interchangeable among the five plants. The areas in the plane of the rack faces above the effective height are covered with timber bulkheads to minimize the formation of ice on the rack bars as described elsewhere herein. Furthermore, a concrete curtain wall upstream of the stop-log slots, is carried down to one foot (1') below normal drawdown elevation. These plants are provided with mechanical rakes which, at times are used to dislodge frazil ice or anchor ice.

Fig. 3 is a typical mill type rack with sloping bars and trash baffle extending below the minimum water level.

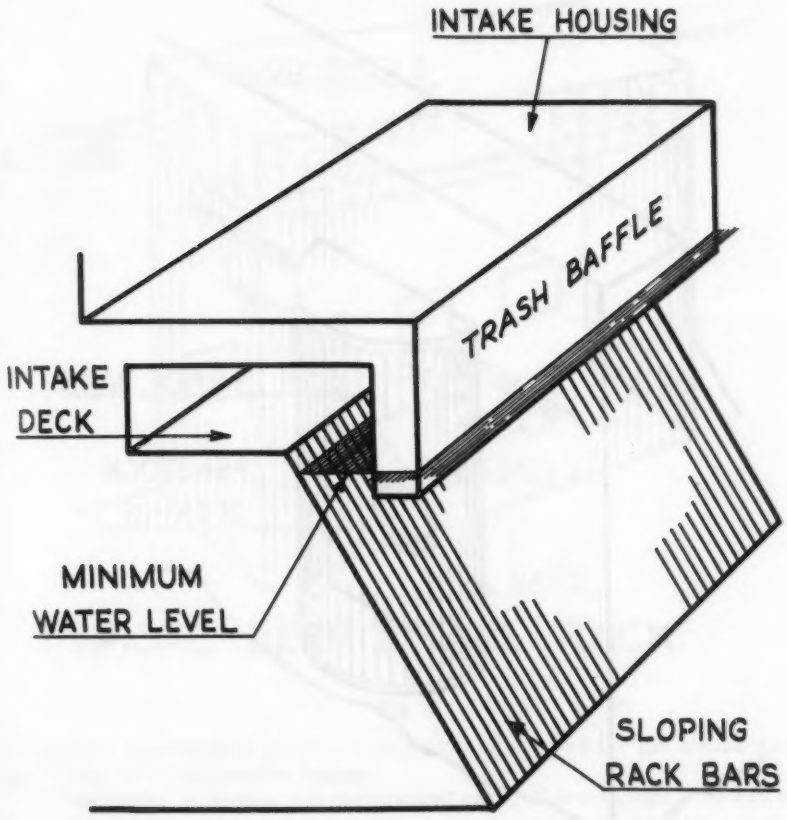
In some cases, an air bubbling system is effective. Compressed air, released through small orifices in a system of air pipes located on the bottom of the forebay, has proven to be quite effective in preventing the formation of surface ice in front of the rack structure. The air discharging orifices should be located sufficiently upstream from the face of the racks as to cause the bubbles to reach the water surface a few feet in front of the rack bars. The relatively wide range of approach velocities obtaining at any one intake, depending upon the water demand of the turbines, requires careful analysis in the determination of this location.

One of the many variations found in the design of trash racks is the inclination of the rack face. Some designers prefer the vertical trash rack while others prefer the sloping rack. The arguments for the sloping racks are usually predicated upon the idea that trash rakes will ride more successfully on a rack which is farther from the face of the intake at the bottom than it is at the top. This batter tends to hold the trash rake against the racks by gravity and can usually be provided without appreciable sacrifice of economy, where the height of the racks is relatively low. Deeply submerged racks, such as basket or semi-cylindrical rack structures, where no raking is contemplated, obviously require vertical bars. In the case of trash racks extending to depths between 25 and 125 feet, vertical racks appear to be most economical.

Fig. 4 shows a typical basket type rack with semicircular straight bars enclosing the penstock opening.

Fig. 5 is a typical deep intake rack with vertical face enclosing several penstock openings as a combined structure resting on the bottom of the dam with straight end faces.

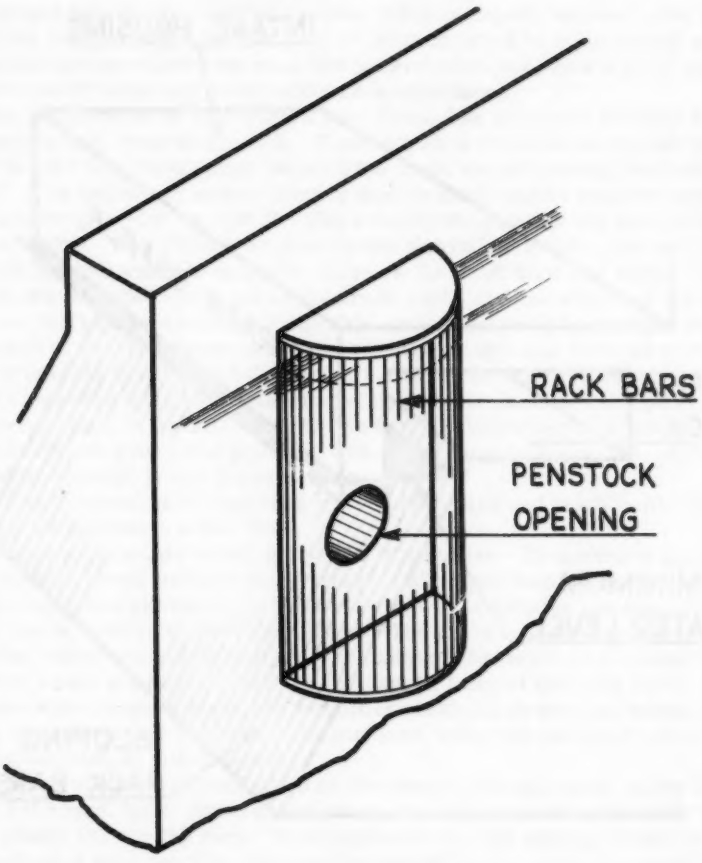
The condition commonly occurs wherein it is advisable to provide a mechanical trash rake to enable the operating personnel to clean the trash racks of debris from time to time. With an adequately designed trash rake, properly used, it is possible to decrease the differential head for which the racks must be used. Only small trash in moderate quantity can be hand raked. The maximum depth for hand raking is about twenty feet and is seldom satisfactory except with sloping racks.



TYPICAL MILL TYPE RACK

Fig. 3

One of the most difficult problems in connection with trash rack design is the provision of a trash rake which will prove to be satisfactory in operation. The difficulty seems to be, not in the inability of the trash rack manufacturers to provide a rake which will perform a certain function, but in the proper specification of the type and amount of trash which the rake must handle. The quantity of the trash to be handled as well as the ability of this trash to withstand the raking action is usually under-estimated. In addition to the apparent tendency to under-design a trash rake, there is also apparent an inclination on the part of many designers to over-complicate the mechanism. This last is readily understood in view of the fact that not only must the rake be capable of an up and down motion but the position of the teeth must be changed from vertical, while the rake is travelling downward, to the horizontal, when the

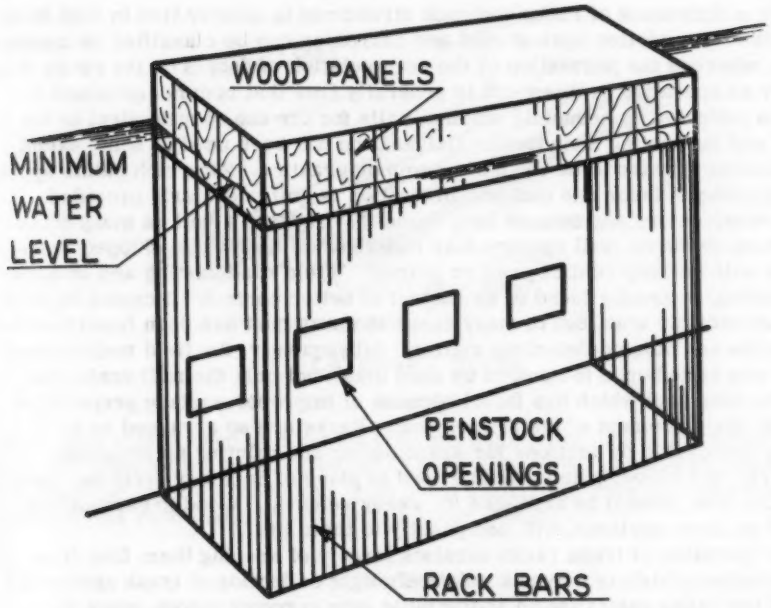


TYPICAL BASKET TYPE RACK

Fig. 4

rake is being lifted. Furthermore, provision must be made for moving the teeth to release the rake from a load too great for the rake to handle.

One of the most troublesome features of trash rake design, insofar as the operator is concerned, is the rake apron and collection hopper. If one is faced only with small twigs, leaves, and other debris having no dimension much greater than one foot, it can readily be raked by hand, from the rake teeth across the apron into the hopper. Similarly, no great difficulty will be found when transferring this type of debris from the hopper to a truck or trash sluice. If, however, and this is more frequently the case, the trash consists of sizeable branches, grasses, vines, small animal carcasses and similar objects, it is only with great difficulty that the interwoven mass can be moved by hand rake even on a smooth horizontal surface. Trash rakes should be so



TYPICAL DEEP INTAKE RACK

Fig. 5

arranged as to provide a rather steeply sloping surface for the discharge of trash from the rake into the hopper.

The handling, by means of a mechanical rake, of trees, logs, timbers and large animal carcasses, none of which are supposed to get past the log boom, but some of which do get past, is seldom possible to the satisfaction of the operator. Some mechanical rakes with log hooks are quite effective in handling relatively short, regular logs or timbers. A few branches, or a cross arm or similar addition to a simple timber, however, will do much toward making the log hooks ineffective. Such material is usually best handled by a truck crane with a clam-shell bucket or log tongs operating from the intake deck.

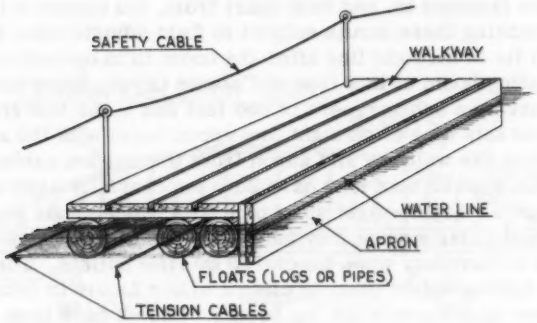
In many locations grass-like plants consisting of long tendrils make it difficult to keep the racks clear, even with a normally adequate mechanical rake. In some cases these grasses grow on the bottom of the reservoir in shallow water and are pulled loose by the ice during periods of thawing. Several methods of handling this type of debris have been used, varying from the questionable practice of pulling part of the racks in the expectation that the grass will safely pass through the wicket gates and runner, to cutting the grass with knife-like blades on pike-poles. Some racks have been made of pipes, instead of bars, arranged so that the top ten (10) feet of every other pipe could be removed. The fixed pipes have been slotted and permit the insertion of a pike-pole having a blade affixed in such a manner as to protrude through the slot and cut the grass as the pike is forced up and down in the slotted pipe. Several plants on the upper Snake River, Idaho have been so equipped.

The maintenance of racks and rack structures is closely tied in with their operation. Protection against rust and corrosion can be classified as maintenance, whereas the prevention of the accumulation of debris on the racks is in reality an operating problem. It is generally true that common practice today, as indicated by economic studies, calls for the use of mild steel in the racks and supporting structures. Galvanizing has been used to some extent, but painting appears to be the more common practice. Zinc rich paints appear to be gaining in favor and cathodic protection is quite regularly provided. Some constructors and owners have found that racks, as well as many other structural features, will require less maintenance if they are shipped to the jobsite with no shop coat of paint or primer. Field sandblasting and immediate painting is usually found to be subject to better control. It cannot be said to be universally true, but in many cases the shop coat has been found to cover mill scale and other deleterious matter. Subsequently, the field maintenance crews are called upon to remove by sand blast, not only the mill scale, but also the shop coat which has failed because of improper surface preparation prior to the placement of the primer coat. Racks are so arranged as to be readily removable by sections for sandblasting and painting as required. Similarly, the timber panels, where used in place of rack bar sections above the water line, should be arranged for ready removal. Wood preservatives, if used on these sections, will add greatly to their life.

The operation of trash racks consists largely of keeping them free from an accumulation of debris. Even a relatively light collection of trash against the face of the racks can cause an appreciable loss in power output, since the head loss through the racks varies as the square of the water velocity. A relatively small increase in head loss can cause the loss of many thousands of dollars worth of power or energy in the course of a year. Furthermore, it can be pointed out that the money spent for each foot of incremental head on the dam may run into hundreds of thousands of dollars. As described elsewhere herein, only very low head intakes can be kept clean by hand raking. The present day high cost of labor tends to decrease the height, or overall size, of the intake which will be found economical to clean by hand. Some types of trash can not be handled either by hand raking or mechanical raking. In some cases, where the rack velocities are exceptionally low, some of the heavier forms of debris, if not water-logged, can be snaked by tag-lines to a sluice or other point of disposal. Devices for handling other specific types of trash are described elsewhere herein. The main objective in trash rack operation is to keep the racks clean so that the head lost because of trash is of less value than the cost of cleaning the racks.

Mechanical trash rakes are usually somewhat complicated in nature and thus require careful maintenance. Many rakes suffer periodically from bent teeth or log hooks. These can frequently be strengthened by the welding on of reinforcing members. Like all wire rope, the hoisting and tooth positioning cables require regular inspection and greasing. All sheaves should also be inspected regularly to avoid the development of worn spots which in turn will adversely affect the life of the wire rope. Motors for rake travel and hoisting, as well as all moving parts, require careful observation, largely because of their exposure to the weather.

The needed operation of the trash rake has largely been covered previously. One of the most prevalent faults found in mechanical rake operation is the attempting of cleaning tasks beyond the design limitations of the rake. This, for example, explains the bent teeth often encountered. The operator of a



CATENARY BOOM

Fig. 6

trash rake should keep in mind the type and quantity of debris for which the designer planned the rake. A careful study of the purchase specifications, by the operator, may dissuade him from over-loading the rake and subsequently charging the rake manufacturer with a faulty design.

Log Booms

Almost every hydroelectric plant has its own requirements as to the magnitude and type of log boom required. In some situations, the stream or lake impounded carries so little trash that no boom is required. The type of trash, the quantity, the frequency and duration of trash carrying periods of the river all enter into the determination of the details for the boom design. In locations where the trash consists of many types of flotsam, from small bits of brush to large logs or trees, the requirements of a boom are quite extensive.

It is desirable, whenever possible, to have the boom so arranged as to cause the trash to move along the boom by the force of the normal flow of the stream. In order to accomplish this, the boom must be placed at an angle no greater than about 30° with the direction of the surface currents. The face or apron on the upstream side of the boom must be such as to present a flat, vertical surface along which the trash can slide easily. A boom consisting of a series of logs fastened together end to end will do little to prevent trash from being carried underneath and past the boom whenever the velocity past the boom becomes appreciable. It has been found that from two to three feet of submergence of vertical face must be provided, even in low velocity water, to prevent the passage of trash underneath the boom. Even with this arrangement, it is generally necessary to provide a walkway along the boom to permit operating personnel to keep the boom clear of snags. This automatically necessitates the use of two or more logs laid parallel to each other in framing the log boom, or some alternative method of securing the same result.

Fig. 6 shows a catenary boom consisting of floats attached to tension cables with the walkway above the water line and diverting apron.

If the face of the boom can be maintained in a straight line despite the action of the current and the load of the trash against the boom, the action of the boom in carrying the trash along its face to the downstream end will be much improved. This can be readily accomplished in a boom constructed of logs or similar floats tied together with a tension cable by carrying the tension cable, in its normal catenary curve, on one set of logs or floats while the apron face and catwalk is carried on a second set of floats. The walkway and

apron floats are fastened to, and held apart from, the catenary floats by means of struts. By making these struts subject to field adjustments, the apron face can be made to lie in straight line after the boom is in operation. The apron face, in an existing boom of this type at Cabinet Gorge, Idaho consists of two inch planking having a submergence of two feet and a one foot freeboard.

Fig. 7 depicts this type of straight line apron boom with the adjustable struts to separate the walkway and apron from the tension cables.

Many materials other than logs have been successfully used in the construction of booms. Light weight steel pipe with welded ends have been used successfully, particularly when they have been filled with styro-foam in order to prevent loss of buoyancy when punctured by rifle bullets. The geographical location of the hydroelectric plant is often a major factor in determining the economies of use of materials for log booms. Booms have been constructed of metal tanks or Navy type floats or pontoons.

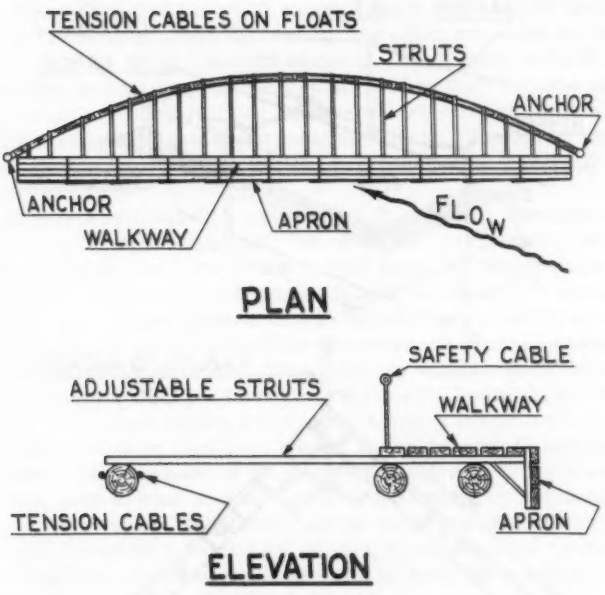
One such boom has been designed and constructed at the Noxon Rapids plant in Montana. This is a V-shaped boom spanning the five intakes. Each of the two legs of the boom is 350 feet long and is supported as a horizontal catenary by a cable extending from the face of the intake to the apex. From the apex, or nose, a double anchor cable extends 1350 feet to an anchor fastened to bed rock on the reservoir shore. The legs of the boom are made up of rectangular floats or pontoons seven feet long, five feet wide and five feet high. These are ballasted with cement mortar to float with three foot draft and are fastened together with hinges to provide freedom of relative movement in a vertical plane and, through 1/16 inch pin clearances in the hinges, to provide a restricted movement in a horizontal plane. The design sag of the catenary from the chord is eight feet, requiring a pull on the anchor cable of 54 kips to withstand a uniform load of 20 lbs per lineal foot normal to the boom. This boom has been found to be quite stable and presents a solid vertical face for trash deflection with 3-1/2 in. gaps between floats.

Fig. 8 shows a perspective view of the steel pontoon with hinged attachments and plan view of the triangular attachment to the anchor cables.

In some locations subject to cold weather, booms are used to divert the flow of ice from the intakes. Because of its specific gravity of 0.922, ice is one of the most difficult materials to prevent from passing under a boom. The more dense woods, or woods which have become "water-logged" are in the same category except that sheets of floating ice have a tendency to slide under or over that ice which has been stopped by the boom. Thus a condition is reached whereby the bottom sheets of ice are below the bottom edge of the apron and are free to pass under the boom.

In a few rare cases, where the approach to the intake is quite shallow and the individual pieces of flotsam are light, a wall of concrete can be constructed with submerged openings for water passage. This type of guide wall can be supported on piers to the river bed instead of being supported by its own buoyancy. Such a wall, or bottom-shaped boom, has been successful at the Avanhandava Hydroelectric Development in Brazil. At this plant, the wall is placed at 45° to the thread of the approach current to obtain the desired "shearing" or "self-cleaning" action. The debris handled by this wall consists of large quantities of water hyacinths and aguape.

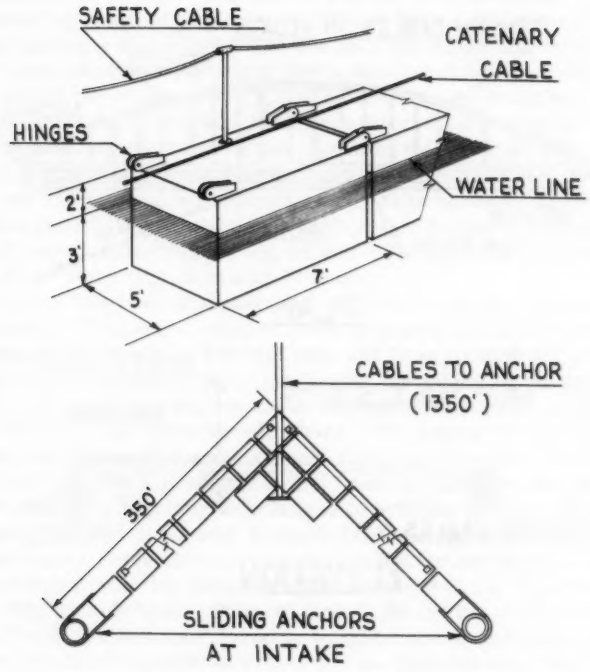
In some situations, it has been found advisable and feasible to provide devices for keeping the trash boom free of accumulating trash. One such device consists of an arrangement of water spray nozzles, as are commonly used to clean traveling screens in steam electric station or water supply intakes. The



STRAIGHT LINE APRON BOOM

Fig. 7

nozzles are attached to the upstream side of the boom and are so directed as to cause the flat jets of water to impinge on the forebay surface in a direction which will produce surface currents to carry the debris along the boom. This arrangement works best with debris consisting of small, light individual pieces. Logs and similar large objects having more draft than leaves, flowers or twigs will be controlled by the currents deeper than the surface currents established by these water jets. Occasionally, this heavier, larger debris has been moved along boom faces by motor driven, submerged propellers mounted along the boom. These propellers develop currents extending sufficiently below the surface as to move debris having up to about twelve (12) inches of draft. Such current producing devices are quite expensive from the standpoint of operation and maintenance and require careful study to justify their installation.



STEEL PONTON BOOM

Fig. 8

Maintenance of trash booms obviously varies as widely as, and in keeping with, the design of the boom in question. The most critical part of most booms is the tension member, that is, the cable, chain or combination of floats and linkages forming the load resisting member. In the case of a catenary boom depending upon a wire rope for support, the cable will be subjected to intense weathering action. Usually wet and frequently exposed to the air, cables must be inspected carefully and regularly. Rust and corrosion are sometimes permitted to weaken the cable so that when an unexpected load, even though less than the original design load, is placed upon it, the cable may fail. Such

failures can prove to be far more expensive than one would suppose. A long boom folding up on itself across a spillway with its gates open and becoming entangled with the piers can prove to be extremely effective in blocking the spillway. A boom thus doubled up at random has a remarkable ability to catch whatever trash is floating down the stream and often presents serious problems by resisting efforts toward removal. Logs used as floats must be carefully watched so that they can be replaced when they become water-logged. Walkways and hand rails or safety-belt wire ropes must be inspected frequently for protection against personal accidents. The apron and its supports are not easily inspected but must, nevertheless, be carefully checked for functional adequacy.

One factor which is frequently overlooked or partially neglected in the case of trash booms is the operating cost. Many times, an operating company will spend relatively large amounts of money each year for the operation of a boom which could be saved by the proper layout and design of boom. Through careful design, even at the expense of a relatively large initial investment, overall economies can be realized through the decrease in operating manpower. Trash booms must be kept free from accumulations of drift. Too large an accumulation may seriously overstress the boom structurally. Short of failure, such an accumulation will tend to cause a large amount of trash to be passed under the boom thus overcoming its usefulness in helping to keep the racks clean. The disposal of the debris collected by a boom varies considerably from case to case and the method is dictated by the design of the entire boom arrangement. Whatever facility is provided, such as sluice ways, beaching areas, temporary catchment booms, trash removal cranes or other devices, this facility should be utilized so as to keep the boom clean and in readiness for unexpected floats of debris or ice.

CONCLUSION

It is recognized that there are many other variations of intakes, racks and booms which are adaptable to special conditions which exist at particular plants. Experience in operation and cost at other plants on the same stream or in the localities where similar problems exist leads to progressive development and techniques in design and methods of handling accumulated debris. The Committee welcomes discussion and contributions of designs which have proved successful in obtaining the objectives of low first cost and minimum maintenance requirements.

Respectfully submitted,

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DESIGN AND SELECTION OF HYPERBOLIC COOLING TOWERS

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SUMMARY

The structural design of hyperbolic cooling towers is discussed with particular reference to wind stresses in the shell. Formulae are given to enable the size of cooling towers to be determined for a given cooling duty, and a method outlined for the selection of the most economic duty.

INTRODUCTION

The first hyperbolic natural draught reinforced concrete cooling tower was designed by Prof. van Iterson of the Dutch State Mines and installed at the Emma Colliery in 1916. Towers of this type were installed at Lister Drive Power Station in Liverpool in 1925 and since then have become standard practice in British power stations where cooling towers are required. Many early towers are still in use, having shown great reliability in service, little maintenance having been required apart from the replacement of decaying timber in the cooling stack. Recent advances in the fields of timber preservation and the use of alternative materials promise to extend the life of the whole of a modern hyperbolic cooling tower to the working life of its associated power station.

The largest tower so far constructed is 340 ft. high and 260 ft. base diameter and will cool the circulating water for a 200 MW set. Most of this structure is the empty shell but the lower portion contains the cooling stack over which the warm water is distributed by a pipe and nozzle system 32 ft. above the ground. The lower portion of the shell is open to allow the air access to the cooling stack, the shell being supported on legs that are inclined to resist the shearing force due to the wind. Beneath the tower a pond is constructed to catch the falling water and return it to the circulating water system. An example of a modern hyperbolic cooling tower is shown in Fig. 1.

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1. Plant Design Branch of the Central Electricity Generating Board, London, England.

The function of the cooling stack is to increase the surface area between the water and the cooling air, either by breaking the water up into droplets or by spreading it over a large area in the form of films. It is important that this be achieved without offering too great a resistance to the movement of air.

As the warm water falls through the stack it gives up its heat to the air. The air leaving the stack inside the shell is lighter than the ambient air and a draught is created by chimney effect. This mechanism differs from that of a mechanical draught tower in that the cooling is dependent on the dry bulb temperature as well as the wet bulb temperature, the draught for a given wet bulb temperature increasing with decrease of dry bulb temperature (i.e. being better in humid conditions). Above the distribution pipes a spray eliminator screen is constructed to catch the fine droplets of water which would otherwise be picked up by the rising air-stream and deposited over the neighbourhood. As the plume of vapour is discharged at a great height there is no

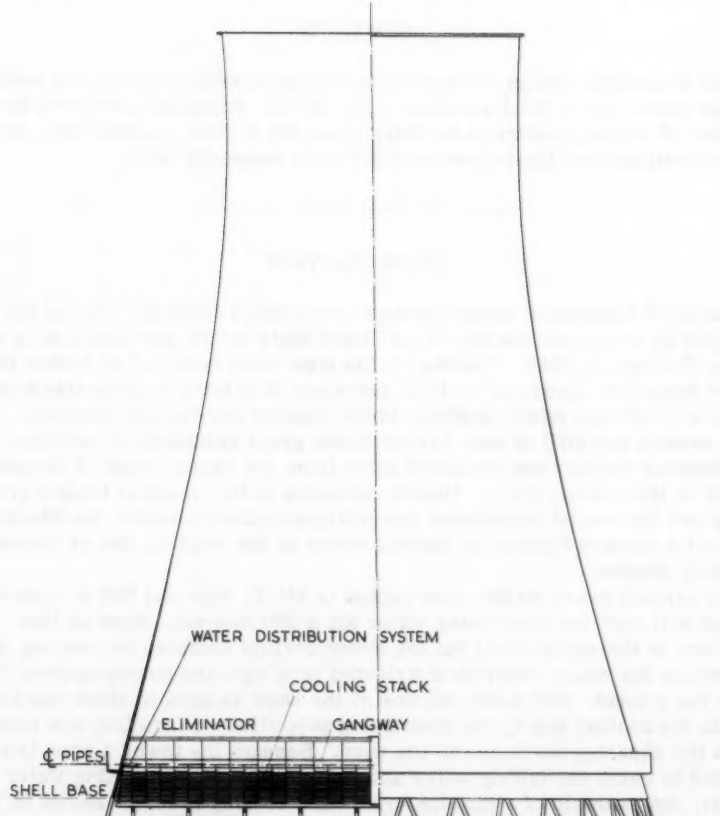


FIG. 1 TYPICAL MODERN NATURAL DRAUGHT COOLING TOWER

nuisance due to ground fog, nor is there any risk of the performance of the tower being affected by the recirculation of the air leaving the top of the tower.

The cooling characteristics of a hyperbolic type natural draught cooling tower are different from those of a mechanical draught installation. If it is desired to compare the economics of the two types of tower it is necessary first to select the most economic cooling water arrangement for each type of tower by a method similar to that outlined in the paper. The most economical arrangements of both systems can then be compared taking into account such factors as overall capital cost, condenser vacuum, auxiliary power required, reliability, cost of maintenance and ground area. It can be quite misleading to compare the economics of the two types of cooling tower on the basis of a common cooling system and tower duty.

Design of Tower Shell by Membrane Theory

There is no thermo-dynamic reason why the shell of a natural draught cooling tower should not be cylindrical. This shape simplifies both the design and construction of the shell and cooling towers have been built in Germany on these lines.

However, the momentum of the air entering the shell carries it into the centre to form a vena contracta whose diameter depends on the ratio of tower diameter to height of air inlet. A considerable saving in shell surface area and volume of concrete can then be made by tapering the shell in to the diameter of the vena contracta. This stiffens the shell against wind forces and opening out the shell above the throat stiffens the shell even more.

The analysis of the effect of wind forces on a shell can be carried out by the "membrane" theory, which assumes that the thickness of the shell is so small compared with its diameter that the wind forces are resisted only by direct tensions, compressions and shearing forces in the direction of the shell itself. The analysis of the true hyperbola of revolution is complex and is usually carried out using a step-by-step method. A very good approximation however can be made by assuming the shell to be made up of two truncated cones with a cylinder in between. Towers of this "diabolo" shape have been built in the past but as they are rather ugly and show no other advantage than ease of calculation they are no longer being constructed. The simple case of the cylinder is dealt with to demonstrate the method, and the analysis of a tower built up of two truncated cones is shown in Appendix 1.

Analysis of Cylindrical Shell

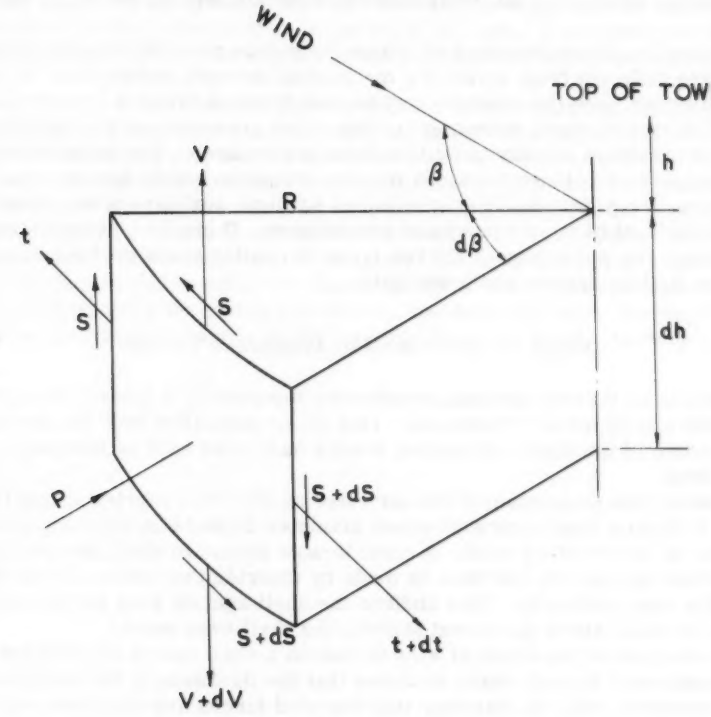
Consider the small element of shell shown in Fig. 2 acted upon by a normal wind pressure p, which is resisted by vertical loads/ft. v and (V + dv), horizontal ring loads/ft. t and (t + dt) and shearing loads/ft. s and (s + ds).

These forces are in equilibrium.

Resolve Vertically

$$v \cdot R d\beta + s\, dh = (v + dv)\, R\, d\beta + (s + ds) dh$$

$$\text{Whence} \quad ds/d\beta + R\, dv/dh = 0 \qquad (1)$$



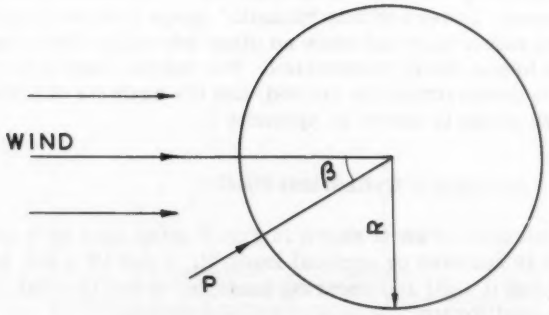


FIG. 2. DIAGRAM OF CYLINDRICAL TOWER

Resolve Horizontally

$$t\,dh + s.R d\beta = (t + dt)dh + (s + ds)\,R d\beta$$

$$\text{Whence} \quad dt/d\beta + R\,ds/dh = 0 \qquad (2)$$

Resolve Perpendicularly to Element

$$t = -pR \qquad (3)$$

Solving these equations will give values of v, s and t at any depth h from the top of the shell and any angle β to the wind.

From (3) $dt/d\beta = -R\,dp/d\beta$

Equation (2) becomes $-R\,dp/d\beta + R\,ds/dh = 0$

$$\text{Whence} \quad ds/dh = dp/d\beta$$

$$\text{And} \quad s = h\,dp/d\beta + \text{const.}$$

At top of shell $s = 0$ (i.e. when $h = 0$ $s = 0$) $\therefore$ const. $= 0$

$$\therefore \quad s = h\,dp/d\beta \qquad (4)$$

$$ds/d\beta = h\,d^2p/d\beta^2$$

Eq. (1) becomes

$$h\,d^2p/d\beta^2 + R\,dv/dh = 0$$

$$\text{or} \quad dv/dh = -h/R \times d^2p/d\beta^2$$

$$v = -h^2/2R \times d^2p/d\beta^2 + \text{const.}$$

At top of shell $v = 0$ (i.e. when $h = 0$ $v = 0$) $\therefore$ const. $= 0$

$$\therefore \quad v = -h^2/2R \times d^2p/d\beta^2 \qquad (5)$$

The three equations are then

$t = -p\,R$ ..... (3)

$s = p'h$ ..... (4) where $p' = dp/d\beta$

$v = -p''\,h^2/2R$ ..... (5) where $p'' = d^2p/d\beta^2$

In order to solve the above equations it is necessary to know how the wind pressures are distributed around the tower shell, i.e. how p varies with β.

Wind Pressure Measurement

Although there have been some experiments on cylinders and model cooling towers to find out the pressure distribution, in no case did the Reynolds Number of the test approach that attained at high winds on a modern hyperbolic cooling tower, the difficulty being that with a model scale of $\frac{1}{100}$ the wind velocity would have to be 7,000 m.p.h. to correspond with a 70 m.p.h. wind at the same pressure. It is known that the vibration of a chimney can extend sub-critical distributions of pressure to Reynolds Numbers greater than that which

would have been expected and for this reason it was desired to obtain readings at the highest Reynolds Number attainable. A model cooling tower 26-1/2" high was built out of sheet metal with pressure measuring points both inside and outside the shell, and tested under pressure (i.e. at increased density and Reynolds Number) in the variable-density wind tunnel belonging to the Aerodynamics Division of the National Physical Laboratory at Teddington, England.

The distribution of wind force obtained from these tests is shown in Fig. 3. Although it would have been more accurate to use one equation to cover the whole curve, in view of the successive differentiating required it was felt that it would be easier and sufficiently accurate to split up the curve into a number of parts each covered by a simple expression, and this has been done. It will be noted that the curve consists of the algebraic sum of the internal and external wind pressures so that the value of p on the front face is made up of one velocity head positive pressure plus about half a velocity head suction inside the tower.

Example

A cylindrical concrete cooling tower shell is 189'4" diameter 219'6" from the top of the ring beam to the top of the tower and 9" thick. What are the maximum stresses produced by a uniform wind speed of 90 m.p.h.?

$$t = -pR = -94.66p$$

$$s = p'h = +219.5p'$$

$$v = -p'' h^2/2R = -255\,p''$$

The maximum values of t and v will occur at $\beta = 0^\circ$ and the maximum value of s at $\beta = 47.6^\circ$.

The wind pressure on a vertical plane surface at right angles to the wind will be $wv_a^2/2g$ where w is the density of the air and v_a the velocity of the wind. With a velocity of 90 m.p.h. a temperature of 59° F and a barometric pressure of 29.53 in. of mercury this will equal 20.75 lb/sq ft.

$$p = 20.75 \times 1.524 \cos 1.89\beta = 31.6 \cos 1.89\beta$$

$$p' = -31.6 \times 1.89 \sin 1.89\beta = -59.7 \sin 1.89\beta$$

$$p'' = -59.7 \times 1.89 \cos 1.89\beta = -113 \cos 1.89\beta$$

$\beta = 0^\circ$ $p_o = 31.6$ $p'_o = 0$ $p''_o = -113$

$\beta = 47.6^\circ$ $p_{47.6} = 0$ $p'_{47.6} = 59.7$ $p''_{47.6} = 0$

t at $\beta = 0^\circ$ = 2990 lb/ft. (compression)

s at $\beta = 47.6^\circ$ = 13,100 lb/ft.

v at $\beta = 0^\circ$ = 28,800 lb/ft. (tension)

A 9 in. shell weighs 112.5 lb/sq. ft. hence

v dead load = 219.5 ft. x 112.5 lb/sq.ft. = 24,700 lb/ft.

The maximum resultant vertical tension is 4100 lb/ft.

If the permissible tensile stress in the steel is taken as 24,000 lb/sq. in. the area of vertical steel required per foot of the shell is 0.17 sq. in. or 3/8" bars at 8" c/c.

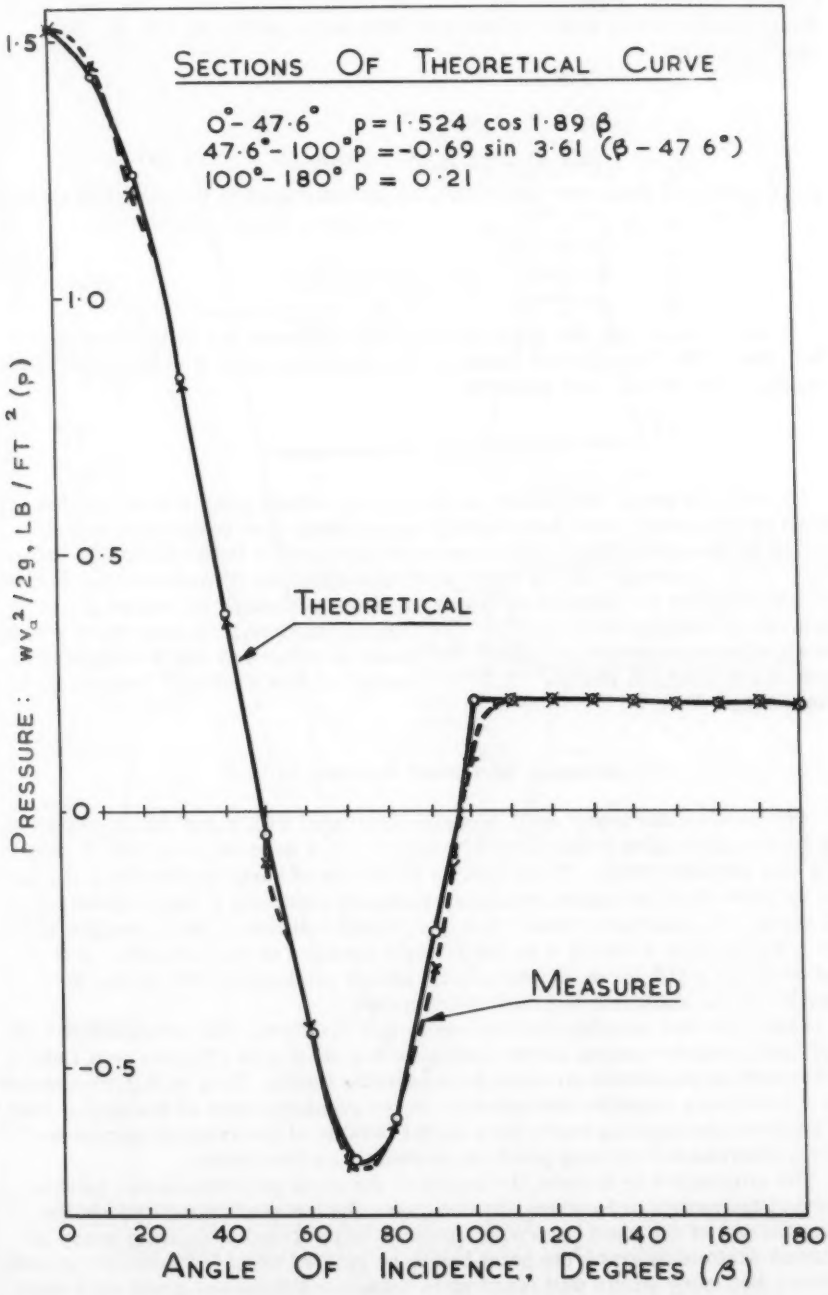

SECTIONS OF THEORETICAL CURVE
0°- 47·6° p = 1·524 cos 1·89 β
47·6°- 100° p = -0·69 sin 3·61 (β - 47 6°)
100°- 180° p = 0·21
1·5
1·0
0·5
0
-0·5
PRESSURE: $wv_a^2/29$, LB / FT ^{2} (p)
THEORETICAL
MEASURED
0 20 40 60 80 100 120 140 160 180
ANGLE OF INCIDENCE, DEGREES (β)

FIG. 3.

PO 5

It is interesting to compare the above results with those obtained for a tower of similar height and radius at the section considered, but made up of two truncated cones with a cylinder in between as shown in Fig. 4. The equations are:

$$t = -98.9\,p$$

$$s = +121.4\,p' + 5.95\,p''' + 0.214\,p^{v}$$

$$v = -28.5\,p - 190.2\,p'' - 18.9\,p^{iv} - 0.63\,p^{vi}$$

which gives the loads per foot with wind pressures as in the previous example.

t at $\beta = 0^{\circ} = -3125$ lb/ft. (compression)

s at $\beta = 47.6^{\circ} = -6143$ lb/ft.

v at $\beta = 0^{\circ} = 13470$ lb/ft. (tension)

It will be seen that the shear and vertical stresses are reduced by over 50% due to the "hyperbolic" shape of the shell compared with a cylinder of the same height and base diameter.

Strain Gauge Measurements

In order to check the validity of the theory, strain gauges were fixed to the shell of the model tower and readings taken under load in the wind tunnel. Owing to the difficulties of temperature compensation in the tunnel the accuracy of the readings was not very great, nevertheless it is considered that the results support the membrane theory as being a reasonable method for the analysis of cooling tower shells. The results obtained from one set of gauges which may be regarded as typical are shown in Figs. 4, 5 and 6 compared with the theoretical results. A fuller account of this work will be published elsewhere.

Moisture Movement Stresses in Shell

In operation the tower shell becomes saturated with water which causes it to swell. According to Murdock[1] a typical 1:2:4 concrete with a w/c ratio 0.6 will expand 0.028%. If the outside of the tower shell is dried by a hot sun or by wind while the inside remains saturated, assuming a linear distribution of strain, the maximum tensile and compressive stresses in the section will be $\frac{1}{2} \times 0.00028\,E$ where E is the Young's modulus of the concrete. If E is taken as 2.5×10^{6} lb/sq. in. the tensile stress produced is 350 lb/sq. in. which may be sufficient to crack the concrete.

There are two possible approaches to this problem. The provision of 0.3% horizontal reinforcement on the outside of the shell with 1" cover will reduce the moisture movement stresses to acceptable limits. This is fairly expensive as it involves a considerable increase in the reinforcement of the shell. Also it involves placing long heavy bars on the outside of the vertical reinforcement, whereas the working platform is usually on the inside.

The alternative is to paint the inside of the shell with bituminous paint to prevent the moisture reaching the concrete. As the scaffolding used in the construction of the shell is moving upwards fairly rapidly, to avoid heavy additional scaffolding costs the paint has to be applied while the concrete is still green. Not many paints will stand up to these conditions and great care must

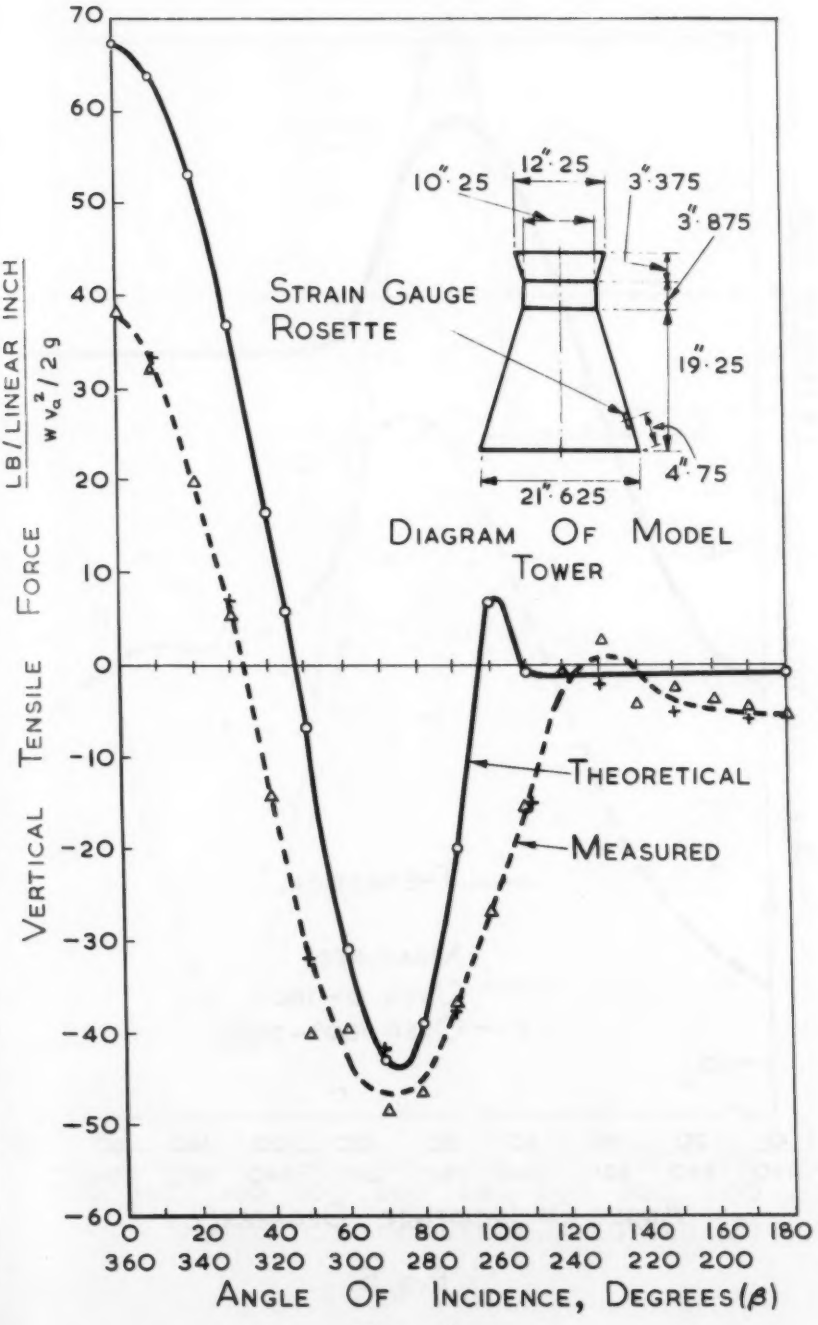

VERTICAL TENSILE FORCE LB/LINEAR INCH $w v_a^2/2g$
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
0 20 40 60 80 100 120 140 160 180
360 340 320 300 280 260 240 220 200
ANGLE OF INCIDENCE, DEGREES (β)
THEORETICAL
MEASURED
STRAIN GAUGE ROSETTE
10".25
12".25
3".375
3".875
19".25
4".75
21".625
DIAGRAM OF MODEL TOWER

FIG. 4.

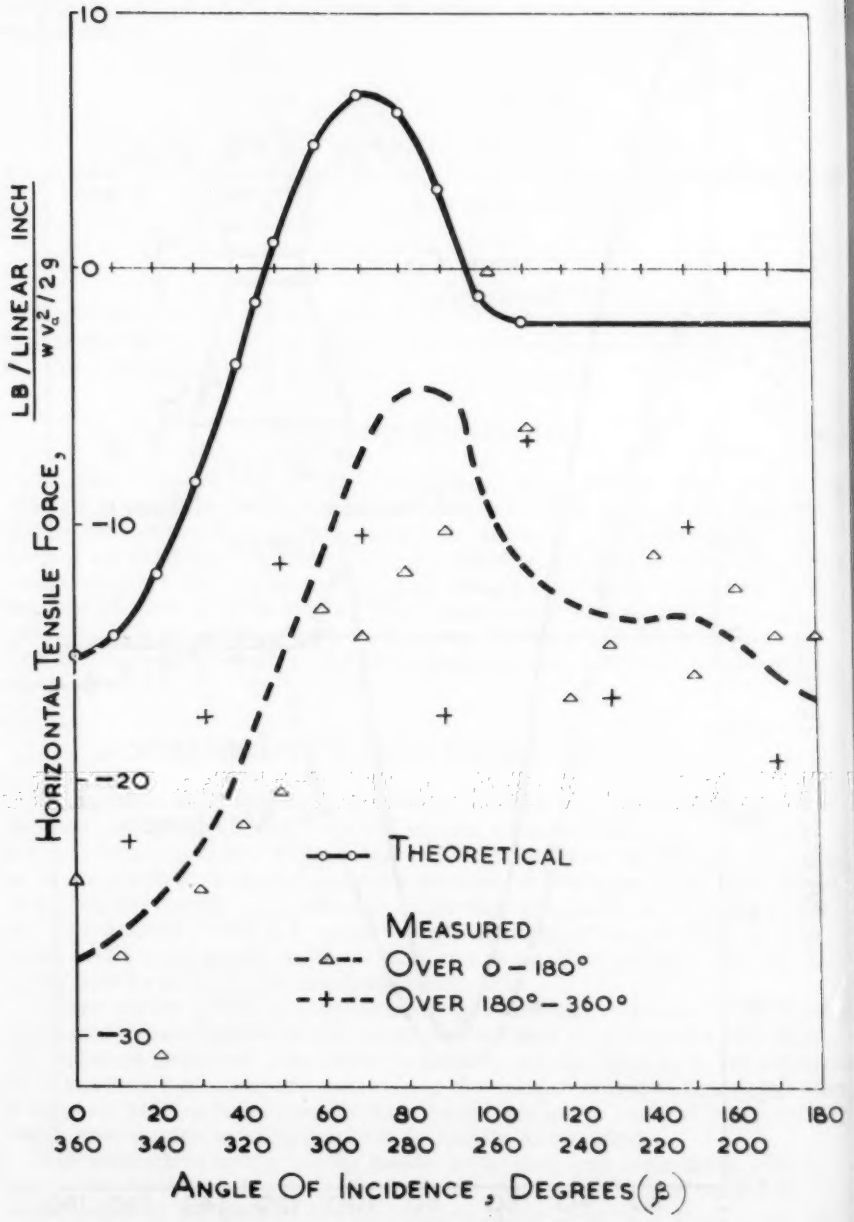

10
0
-10
-20
-30
HORIZONTAL TENSILE FORCE, LB/LINEAR INCH $\frac{w v_a^2}{29}$
THEORETICAL
MEASURED
OVER 0 - 180°
OVER 180° - 360°
0 20 40 60 80 100 120 140 160 180
360 340 320 300 280 260 240 220 200
ANGLE OF INCIDENCE, DEGREES (β)

FIG. 5.

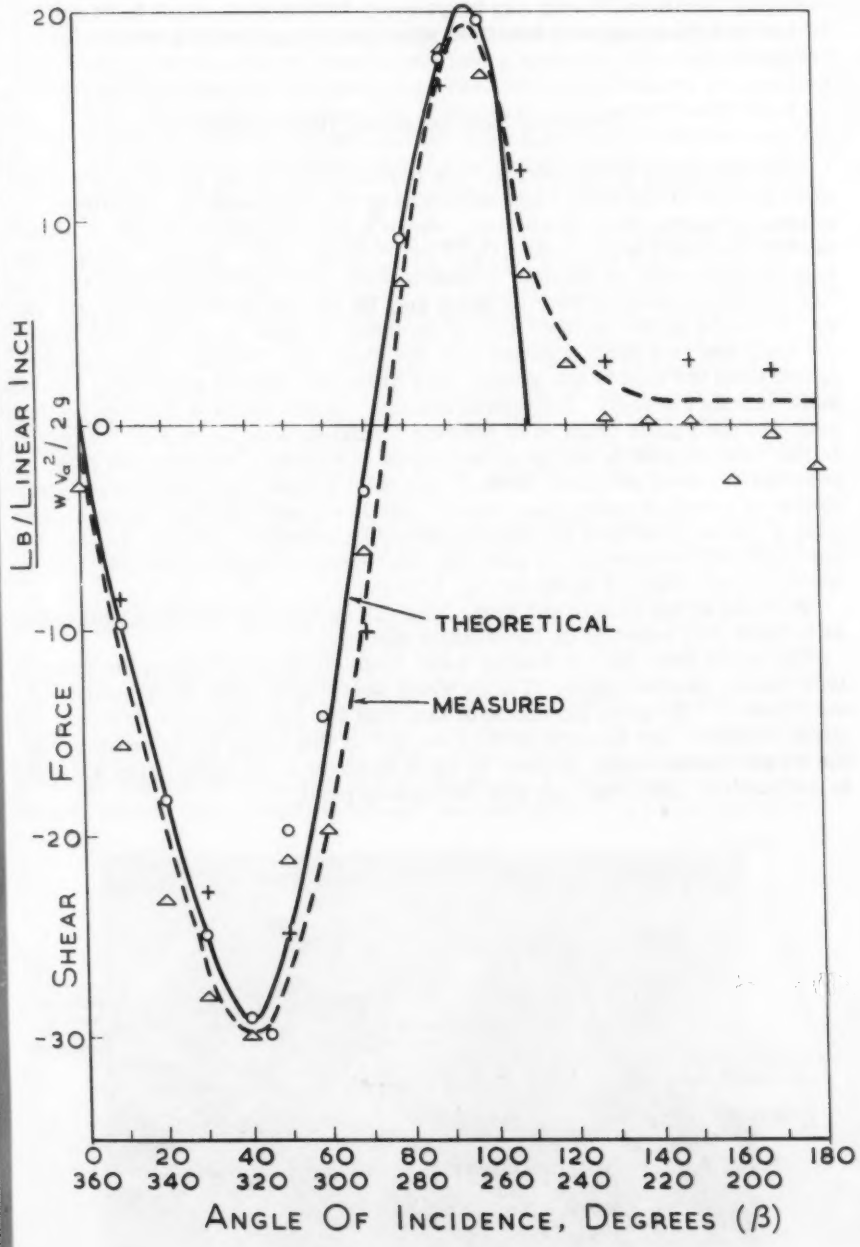

20
10
0
-10
-20
-30
LB/LINEAR INCH
$w V_a^2 / 2g$
SHEAR FORCE
THEORETICAL
MEASURED
0 20 40 60 80 100 120 140 160 180
360 340 320 300 280 260 240 220 200
ANGLE OF INCIDENCE, DEGREES (β)

FIG. 6.

be taken in the selection and application of a suitable paint. Of a large number tested by Parkinson[2] only two proprietory brands were found to be satisfactory and these are now specified where painting of cooling towers is required.

Design of Ring Beam and Tower Legs

The ring beam is the intermediate portion between the tower legs and the main portion of the shell. The thickness of this ring beam is sometimes gradually tapered from the bottom, where it has to be thick enough to accommodate the tower legs, to the 4-1/2" or 5" of the shell. In other designs the ring beam is made of constant thickness with an abrupt transition to the shell. The stresses set up in the ring beam may be calculated from deep beam theory. The load on the inclined legs of the tower is made up of the dead load of the shell and ring beam together with the shear and vertical wind loads obtained from the membrane theory. The tower legs should preferably be in the same line as the shell. Sometimes however, where space is limited, the inclination of the legs is changed to produce a smaller pond as in Fig. 7 Tower B. In this case tension is set up at the change of direction and although it is possible to insert sufficient steel at this point to take the stress there is some danger of cracking taking place due to tensile stresses in the concrete. This danger can be overcome by changing direction gradually instead of abruptly, the length of the curve being such that the tensile strength of the concrete is not exceeded. This is shown in Fig. 7 Tower C.

The load at the base of the tower legs can be taken on the pond wall acting as a lower ring beam or on independent footings.

The spray from the circulating water tends to evaporate from the tower legs leaving concentrations of salts which sometimes cause deterioration of the concrete. To allow for attack of this kind the legs should be designed with adequate cover, the strength of the concrete in this cover being neglected in the stress calculations. If there is much sulphate in the circulating water it is advisable to paint the legs with bituminous paint.

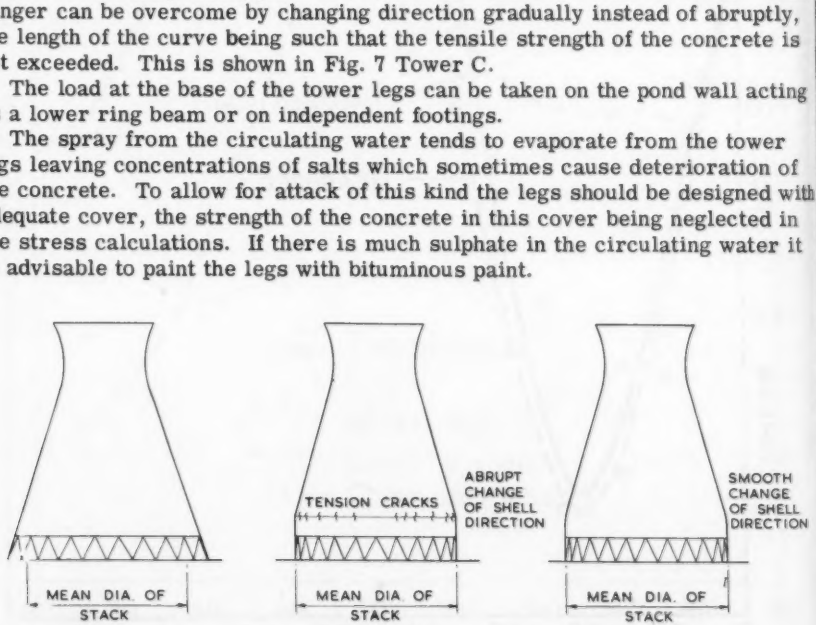


FIG. 7.

Cooling Stack

Before the war timber was universally used as the supporting structure for the cooling stack. Due to decay at the joints, however, the timber stacks gradually became unstable and the structures have usually had to be replaced once or twice during their working life of 30 years. Owing to the shortage of suitable timber during the war precast concrete supporting structures were brought into use; it was then found that the extra capital cost was more than compensated for by the longer life. An example of a precast concrete supporting structure is shown in Fig. 8.

Timber (Pinus Silvestris) is usually used for the splash bars or lathing and a good deal of research has been carried out by the C.E.G.B. and the timber treatment firms into the cause and prevention of decay. Suitable water borne preservatives which combine with the timber to form insoluble salts have given results which promise a considerable extension of life to cooling tower timber when used with the necessarily high degree of impregnation.

The other approach to the problem of packing decay is the use of a packing which does not deteriorate. Glass lathing has been investigated and a highly efficient tower using this material could no doubt be constructed. However, there is an understandable reluctance to use glass on a big scale because of the danger of breakage and falling lathing.

Asbestos-cement sheets as a packing material have also been studied and appear promising. An experimental packing of one layer of corrugated sheets was installed in a small cooling tower in 1945, again stimulated by a shortage of timber. As insufficient depth had been provided this tower gave a disappointing performance but the packing has stood up to cooling tower conditions very well, no sign of deterioration having occurred in thirteen years of continual use. Recent analysis has shown that with a double layer of corrugated asbestos sheets a most efficient tower should be obtained. A tower to this design is being installed in association with a 120 MW set at the present time and should be ready for testing in 1959. Although the asbestos-cement cooling stack was expensive compared with a timber stack for the same duty, the smaller size of tower required brought the total first cost below that of the

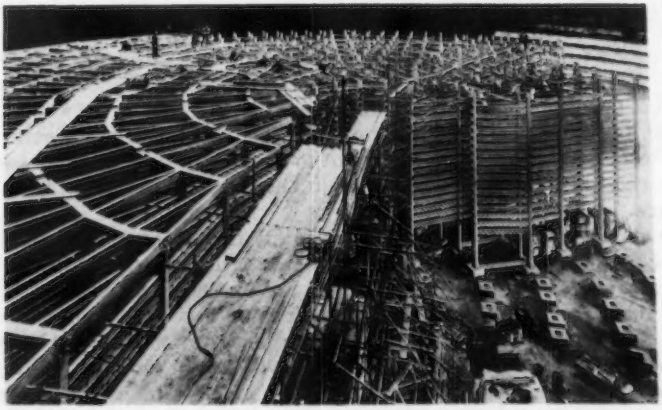


FIG. 8. PRECAST CONCRETE COOLING STACK

cooling tower design with the conventional timber stack. Asbestos-cement packed towers using one layer of flat sheets have also been built in Germany to the design of Dr. W. Otte.

The asbestos-cement packed tower (together with some modern developments in timber packing) has its stack enclosed entirely in the shell, the tower beneath the air opening being empty except for the columns supporting the stack. This has the advantage that the cooling takes place in counter-flow and enables the thermal design theory to be employed with greater accuracy than when some cross-flow takes place. This does not imply that the performance of mixed flow towers cannot be assessed, as a great deal of empirical knowledge has been gained from past experience, but it is simpler to predict the effect of innovations in a counter-flow tower. All counter-flow towers depend for their efficiency on a low overall resistance to air flow and it is important that an adequate air-opening should be employed.

Icing

In cold weather the droplets in the outer periphery of the tower tend to freeze, icicles build up on the packing and sheets of ice form which can damage the packing and obstruct the air flow. In most towers anti-freeze systems are now installed so that hot water can be drawn from the inlet pipe and sprayed over the periphery. A considerable quantity of water is required in really cold weather (probably about 25% of the circulation) and in some cases the capacity of the anti-freeze system has proved insufficient. Additional water can usually be obtained in a centre feed tower by opening the ends of the distribution pipes whereupon the water will run down the tower wall on to the periphery of the stack.

If a counter-flow packing is employed there is no hold for the ice to build up on and serious icing is unlikely to occur.

Make-Up and Purge

During the operation of a cooling tower a small proportion (about 1%) of the circulating water is continually being evaporated and has to be replaced. The salts in the water are left in solution and if the water is not purged concentrations will be reached at which precipitation will occur in the form of scale. Additional make-up water is therefore required above the evaporation loss, the surplus usually spilling over the purge weir to waste. If the purge quantity is twice the evaporation loss the concentration of salts cannot increase to more than 50% above that of the make-up water. Chlorination is also required in certain circumstances to keep algae from growing on the splash bars.

Where an installation rejects its heat solely to cooling towers and the make-up water contains silt which may cause abrasion to condenser tubes, the cooling towers ponds form a useful settlement area. The ponds are therefore designed so that a reasonable quantity of silt can be collected and removed.

Where a limited cooling source is supplemented by towers and the water is silty, saucer-shaped ponds are employed with outlet troughs running across the diameters. Water falling from the cooling stacks will then wash any silt into the outlet troughs and keep the settlement in the ponds to a minimum.

Spray Eliminators

The finer droplets of water produced by the nozzles and the cooling stack are picked up by the rising air-stream and can create a public nuisance if not intercepted. The problem was not tackled effectively until the nationalisation of the British electricity supply industry in 1948 when a large number of eliminator screens was examined in a test tower and the most suitable selected as a standard eliminator screen to be employed in power station cooling towers. This double layer louvred screen shown in Fig. 9A is low in first cost, effective in practice and has the low pressure drop of only three velocity heads. As the column of air inside the tower is lightened by the elimination of droplets no deterioration in performance is produced by the introduction of an eliminator screen of this type. An account of this work has been given by Chilton.(3)

A new development has been the asbestos-cement eliminator screen shown in Fig. 9B for which a British patent is pending. This is more effective, has a lower pressure drop, and is more durable than the timber screen, but is rather more expensive in first cost.

Performance of Cooling Towers

Chilton(4) showed that the duty coefficient D of a tower is approximately constant over its normal range of operation and is related to the tower size by an efficiency factor known as the performance coefficient C as follows:

$$D = \frac{A\sqrt{H}}{C\sqrt{C}} \qquad (6)$$

where A is the base area of the tower measured at pond sill level and H is the height of the tower measured above sill level.

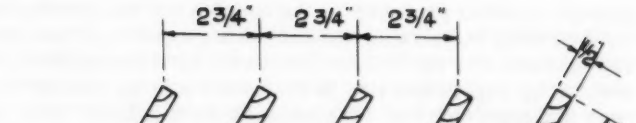
The duty coefficient may be worked out from the formula

$$W_L/D = 90.59 \frac{\Delta h}{\Delta T} \sqrt{\Delta t + 0.3124\,\Delta h} \qquad (7)$$

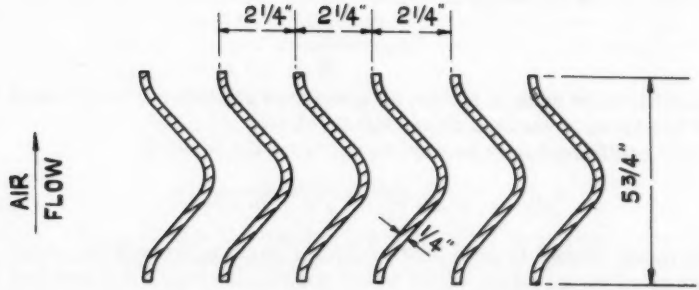
where Δh is the change in total heat of the air passing through the tower, ΔT is the change of temperature of the water passing through the tower and W_L is the water load in lb/hr. The air leaving the packing inside the tower is assumed to be saturated at a temperature half way between the inlet and outlet water temperatures. A divergence between theory and practice of a few degrees in this latter assumption does not significantly affect the result as the draught component depends on the ratio of the change of density to change of total heat and not on the change of temperature alone.

Example

Temperature of water to tower		=	82°F.
Recooled water temperature		=	70°F.
Temperature Range ΔT		=	12°F.
Dry bulb air temperature t_2		=	57°F.
Aspirated wet bulb air temperature t_2'		=	51.7°F.
Water loading to Tower W_L		=	38.2 million lb/hr.
$t_1 = (82^\circ + 70^\circ)/2$	76°	h_1 = 32.1)	from hygrometric
t_2 =	57°	h_2 = 13.6)	tables
Δt =	19°	Δh = 18.5	
$0.3124\Delta h$ =	5.8		
	24.8		



(a) LOUVRE ELIMINATOR



(b) ASBESTOS CEMENT ELIMINATOR

FIGURE 9

$$W_{L/D} = 90.59 \frac{\Delta h}{\Delta T} \sqrt{\Delta t + 0.3124 \Delta h}$$

$$= 90.59 \times \frac{18.5}{12} \sqrt{24.8} = 696$$

$$D = 38,200,000 \div 696 = 55,000$$

The performance coefficients usually attained in the past have been in the region of 5.2 where water loadings are over 750 lb/hr/sq. ft. though new types of packing are bringing this down, i.e. improving it.

Taking a C value of 5.0 and a tower height of 320 ft. the base area of the tower will be $55,000 \times 5 \sqrt{5} \div \sqrt{320} = 34,600$ sq. ft. This means that the internal base diameter at sill level will be 210 ft. A ratio of height to base diameter of 3:2 is normally employed.

In order to find how a tower of any given duty coefficient will perform under varying conditions of air temperatures, water loadings and temperature ranges Eq. (7) has been plotted as a nomogram in Fig. 10.

A more detailed version of this nomogram or performance chart may be modified for a particular value of duty coefficient and included in the specification for a cooling tower. After the tower is constructed it can then be tested within the specified range of conditions and its success judged from the relation between the recooled temperature attained and that shown on the nomogram for the air temperatures, water loading and temperature range imposed on the tower.

The C.E.G.B.'s present practice requires the guarantee to cover water loadings between 90% and 110% of the normal water rate, cooling ranges between 2° F below and 2° F above the normal cooling range, atmospheric wet bulb temperatures between 40° F and 60° F and humidities between 50% and 100%.

It is standard practice to measure the air temperatures at 4 ft. above the ground with an aspirated psychrometer and on this basis towers appear to vary considerably in their performance from one test to the next. As the air

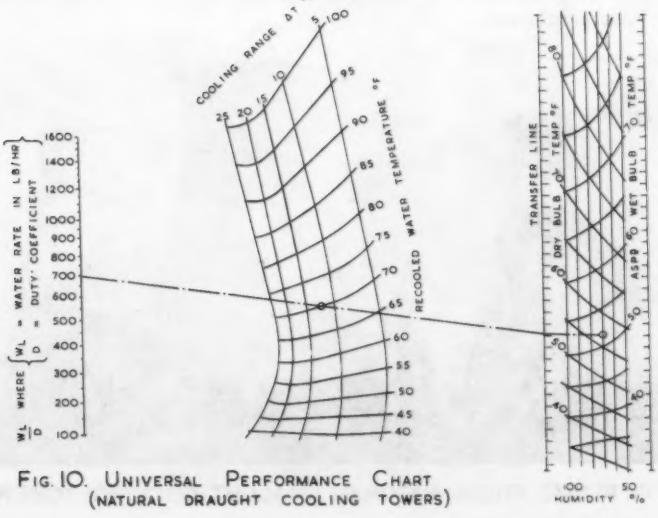


FIG. 10. UNIVERSAL PERFORMANCE CHART (NATURAL DRAUGHT COOLING TOWERS)

RECORD OF TEST ON COOLING TOWER

TEST Nº	WEATHER	TIME	W_L LB/HR	ΔT °F	RECOOLED TEMP °F	PSYCHROMTR HEIGHT FEET	WET BULB TEMP °F	DRY BULB TEMP °F	DUTY COEFFICT D
1	LIGHT S.E. WIND SKY OVERCAST WITH CLEAR INTERVALS	2 - 3 (AFTERNOON)	45×10^6	16·8	68·8	4	48·8	53·0	70,900
						20	47·2	50·9	65,500
2	DITTO	1 - 3 (NIGHT)	44×10^6	15·9	68·5	4	45·3	47·1	57,300
						20	44·9	46·9	56,500

entering the tower actually comes from a higher level than this and the draught is affected by the air density up to the top of the tower it is evident that the more consistent results may be obtained by taking air temperatures at a higher level. The results of two tests taken on one tower are shown in Table 1. More work is continuing on these lines but any rapid change in test procedure is undesirable as most of the test data available have been based on the old method, and the ambient temperature records have also been taken at a standard height of 4 ft. above the ground. Owing to the variation in measured tower duty it is advisable for the tower performance to be assessed on the average duty attained on a number of tests spread over the range of the guarantee.

Wind does not appear to affect the performance of towers significantly, but tests are specified to take place when the wind velocity is less than 15 m.p.h. Towers are tested for not less than four hours with steady operating conditions and the test period is taken as the hour in which conditions are most steady. If the air temperatures are rising or falling the hold up of water in the pond can produce serious errors in the result. Readings are taken every five minutes and all instruments have to have been calibrated not more than six months before the test. Water flows are usually taken by pitot tube traverses of the conduits, but venturis, weirs, and heat balances across the condenser have also been employed.

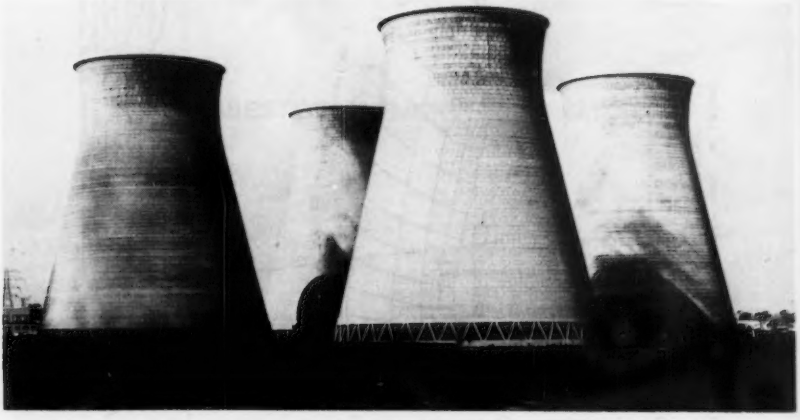


FIG. 11. PLUME FROM NATURAL DRAUGHT COOLING TOWERS

Spacing of Towers

Figs. 11 and 12 show cooling tower installations in modern power stations, the spacing employed and the relation to the rest of the plant. Towers are normally spaced with one and a half internal base diameters between their centres at which spacing they do not appear to influence each other significantly, but further work into this problem is being planned.

Selection of Cooling Water Duty

Introduction

The selection of cooling tower duty cannot be treated in isolation from the rest of the cooling system. Any change in cooling water quantity will be reflected in the temperature range through which the cooling towers have to recool the water and these variations will have repercussions on the capital cost of the other circulating water plant items, namely, condensing plant, cooling water pumps, pipework and valves and also on the cost of pumping the cooling water. This means that the investigation should not only establish the appropriate cooling tower duty, but also the economic water quantity and temperature levels for the complete condensing system.

The factors to be taken into account in assessing the most economic tower duty are (a) atmospheric temperatures, (b) load factor, (c) turbine exhaust characteristic.

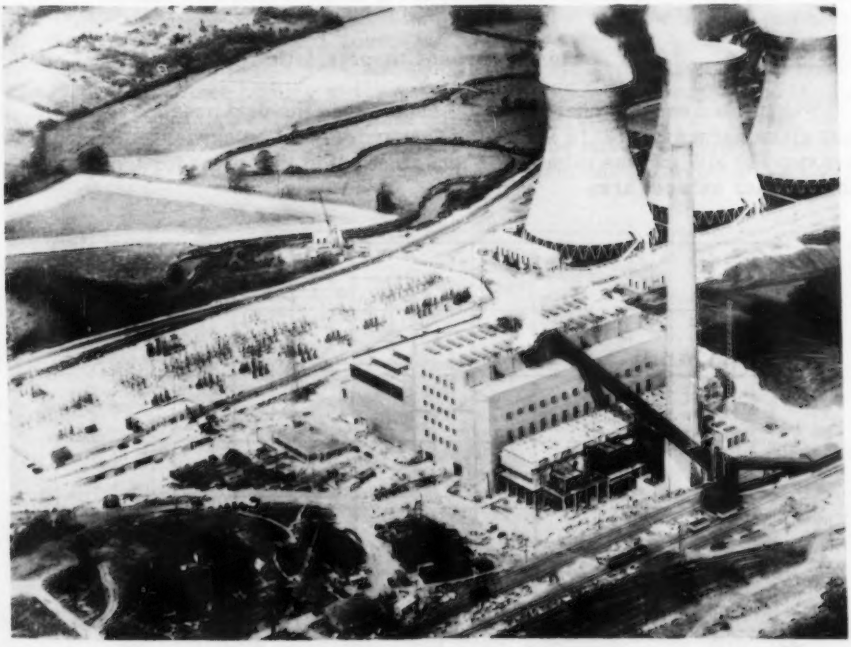


FIG. 12. POWER STATION SITE

Ambient Atmospheric Temperatures

In Britain the atmospheric temperatures do not differ appreciably over the general areas where adequate fuel supplies (and loading) are relatively close at hand, but where cooling facilities may well be limited. The atmospheric temperatures can, therefore, be averaged without seriously prejudicing the result of such investigation. To take an example, for medium load, i.e. 2-shift operation cooling tower installations, the dry and wet bulb temperatures based on a daily period from 6 a.m. to 9 p.m. are significant, and Fig. 13 shows the monthly average of the returns from 13 meteorological stations over a period of 50 years. When considering an installation remote from these general areas of development reference can be made to the local meteorological records.

Load Factor

The plant load factor may also vary and should be given consideration. In Britain the problem is simplified in that machines are expected to be block loaded which means that partial load conditions can be ignored. The average load factor can usually be predicted on an annual basis, but this should, if a more accurate solution is to be obtained, be broken down to a monthly basis. The shape of the load curve is, of course, significant because the most economic design, even for a given annual load factor, will depend on whether generation is concentrated in one period or spread out over the year. Fig. 14 shows the monthly average load factor for a medium load installation.

Turbine Exhaust Characteristic

The turbine exhaust characteristic establishes the performance level for any given vacuum. Fig. 15 shows the vacuum correction curves at full load for two 120 MW turbine exhaust designs, the dotted line being for the one with the smaller exhaust area.

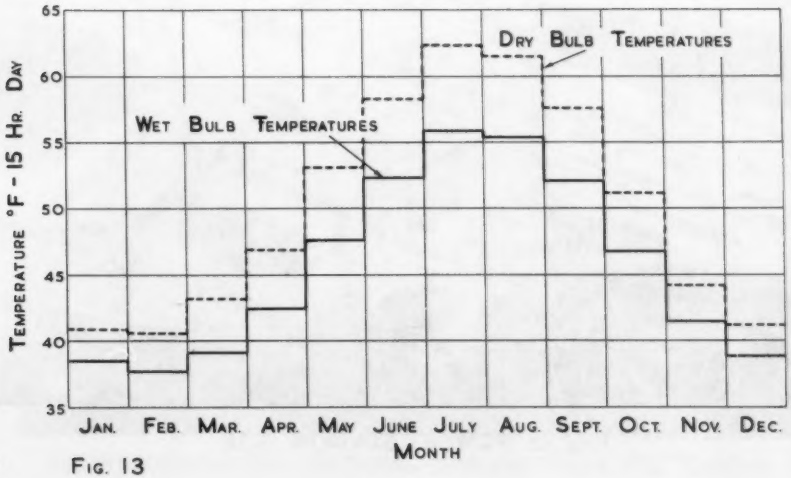


FIG. 13

The average rate of loss in performance for vacua below 28.9" Hg. is 2.4% per inch Hg. for the small exhaust, but there is very little improvement in performance with higher vacua. The large exhaust on the other hand, has an average rate of loss in performance for vacua below 28.9" Hg. of 3.4% per inch, but there is considerable improvement in performance with higher vacua. It will be noted that it can be quite misleading to assume a constant rate of change in performance with variation in vacuum in order to simplify the economic assessment and furthermore the particular turbine exhaust design has a direct bearing on the economics of the cooling system.

Procedure

The method of establishing the economic cooling tower size can perhaps best be illustrated by giving an example of the procedure based on a 120 MW machine.

The monthly averaged atmospheric temperatures and load factors were established and three cooling water quantities namely, 470,000, 587,000 and 783,000 pounds per minute assumed. The total heat rejection from the machine was known and the cooling water temperature range ($T_2 - T_1$) could be established for the various water quantities, T_1 and T_2 being the condenser inlet and outlet cooling water temperatures respectively.

Three sizes of cooling tower having duty coefficients of 72,400 (Tower 1), 58,250 (Tower 2) and 45,900 (Tower 3) were considered in relation to the monthly averaged atmospheric temperatures, cooling water quantities and temperature ranges, in order to establish by use of the performance chart Fig. 10, the averaged monthly re-cooled water temperatures. These temperatures are shown in Table 2 for Tower No. 2.

Three condensing plants were taken into account and were designed to cope with each of the three cooling water quantities and have the necessary heat transfer surface to give final terminal differences ($T_3 - T_2$) of 6°, 10° and 14° F, T_3 being the vacuum steam temperature. The monthly averaged re-cooled temperatures, and the condenser overall terminal differences ($T_3 - T_1$)

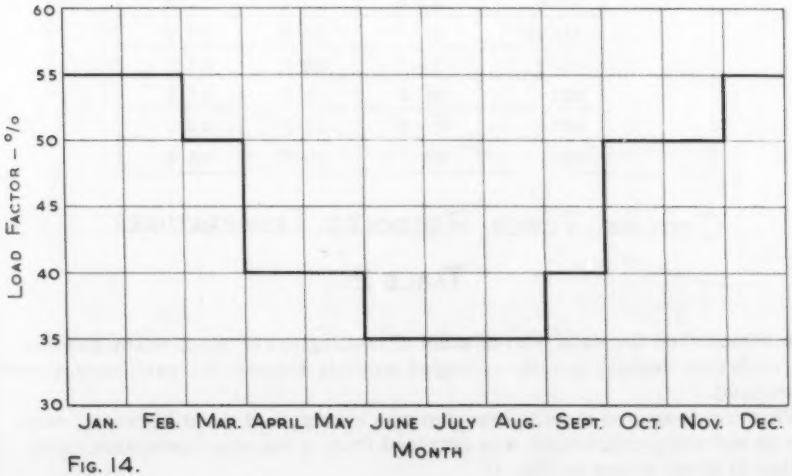


FIG. 14.

	MONTH	DRY BULB TEMP °F	WET BULB TEMP °F	TOWER Nº2 RE-COOLED TEMP °F
$T_3 - T_2 = 6.0°F$	$T_3 - T_1 = 22.0°F$ WATER QUANTITY 587,000 POUNDS PER MIN. $T_2 - T_1 = 16.0°F$			
	JAN.	40.1	38.0	60.4
	FEB.	39.8	37.1	60.1
	MARCH	43.0	38.8	61.1
	APRIL	46.6	41.9	63.0
	MAY	53.0	47.4	66.7
	JUNE	59.2	52.9	70.5
	JULY	63.0	56.2	72.9
	AUGUST	61.7	55.5	72.2
	SEPT.	57.7	52.1	69.9
	OCT.	51.0	46.8	65.9
	NOV.	43.3	40.9	62.2
	DEC.	40.1	38.2	60.5
	$T_3 - T_1 = 26.0°F$ WATER QUANTITY 470,000 POUNDS PER MIN. $T_2 - T_1 = 20.0°F$			
	JAN.	40.1	38.0	58.2
	FEB.	39.8	37.1	58.0
	MARCH	43.0	38.8	58.9
	APRIL	46.6	41.9	60.9
	MAY	53.0	47.4	64.6
	JUNE	59.2	52.9	68.4
	JULY	63.0	56.2	70.8
	AUGUST	61.7	55.5	70.0
	SEPT.	57.7	52.1	67.7
	OCT.	51.0	46.8	63.9
	NOV.	43.3	40.9	60.0
	DEC.	40.1	38.2	58.4

COOLING TOWER RECOOLED TEMPERATURES

TABLE 2

were established for each permutation of cooling tower size, water quantity, and condenser design, and the averaged monthly vacuum for each arrangement established.

When the averaged monthly vacuum was established the differential variation in turbine performance was obtained from a vacuum correction curve similar to those shown in Fig. 15.

The hydraulic losses of the cooling water systems were calculated, and the appropriate differential pumping costs included in the assessment, together with the differential values of turbine performance, and the differential capital cost of the various arrangements.

The cooling systems incorporating condensers designed for a terminal difference ($T_3 - T_2$) of 6° were found to be the most attractive.

The differential total costs of these systems are shown in Table 3 and in Fig. 16. A cooling tower duty D in the order of 60,000 was found to be the economic duty.

This brief outline of the procedure for establishing the economic cooling tower size is based on an installation having no other cooling source than cooling towers, but in some locations it may be attractive to use a small river and supplement its cooling capacity with cooling towers. Under these circumstances the same approach is made to the problem, but the calculations become more complex in that the natural temperature, and water flow variations of the direct cooling source have to be taken into account.

In Britain the complete power station has generally been planned from the outset and the cooling water system has been fully interconnected so that any combination of cooling towers and pumps can operate with any condensing plant. This has been justified on the grounds that when a machine has been out of service the improvement in performance with the remaining machines, due to a better vacuum with all cooling towers in operation or reduced pumping power when relatively low re-cooled temperatures have been available, outweighs the cost of the interconnecting piping and valves. The effects of this feature, however, are not of a sufficient order for it to be necessary to take them into account in the initial economic study.

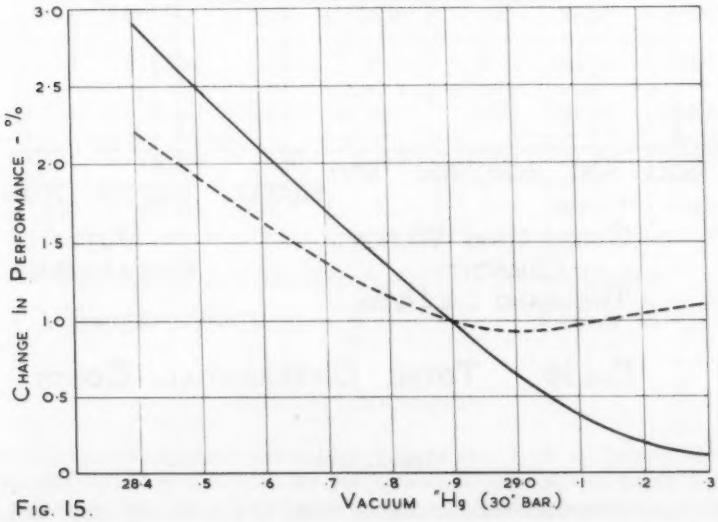


FIG. 15.

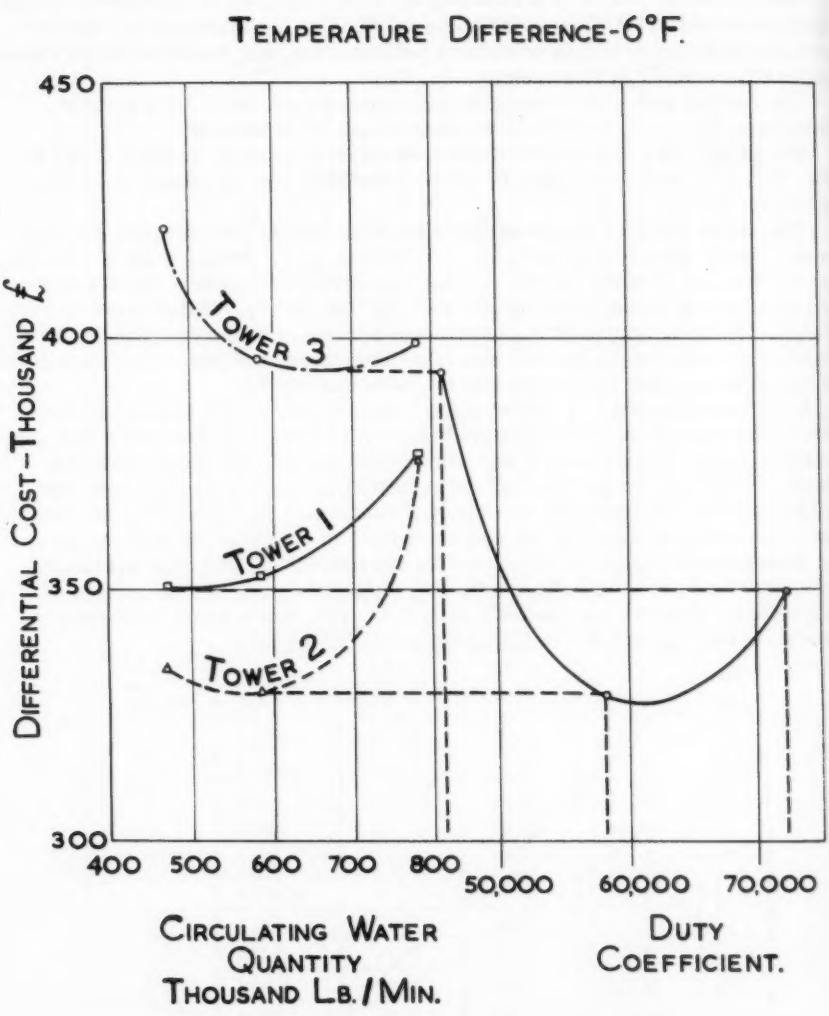


FIG. 16 TOTAL DIFFERENTIAL COSTS

Appearance

As the modern natural draught cooling tower is a relatively large structure it may be necessary to consider a group of towers from an aesthetic point of view and Fig. 17 is included to give some indication of appearance of a cooling installation suitable for 300 MW plant capacity.

TOWER SIZE	COND^R. TERMINAL DIFF. $T_3 - T_2$ °F	WATER QUANTITY 1000 LB. PER MINUTE	TOWER COST + FIXED PUMPING COST - £	VARIABLE PUMPING COST - £	PIPING, CULVERTS, PUMPS & VALVES - £	CONDENSING PLANT - £	TURBINE RUNNING COST - £	DIFFERENTIAL COST - £
1	6	783	328,000	11,460	21,145	15,700	0	376,305
1	6	587	287,700	9,335	8,880	14,200	32,150	352,265
1	6	470	265,585	3,740	0	0	80,750	350,075
2	6	783	254,000	8,350	21,145	15,700	76,050	375,245
2	6	587	203,700	7,000	8,880	14,200	95,850	329,630
2	6	470	181,585	1,870	0	0	150,150	333,605
3	6	783	216,000	5,240	21,145	15,700	140,350	398,435
3	6	587	175,700	4,665	8,880	14,200	191,850	395,295
3	6	470	153,585	0	0	0	267,350	420,935

TOTAL DIFFERENTIAL COSTS OF COOLING WATER SYSTEMS – £ TABLE 3

ACKNOWLEDGMENTS

In conclusion the authors would like to thank Mr. F. H. S. Brown, Member for Engineering of the C.E.G.B. for permission to publish this paper and also their colleagues who have contributed in any way to the preparation of this paper.

PO 5

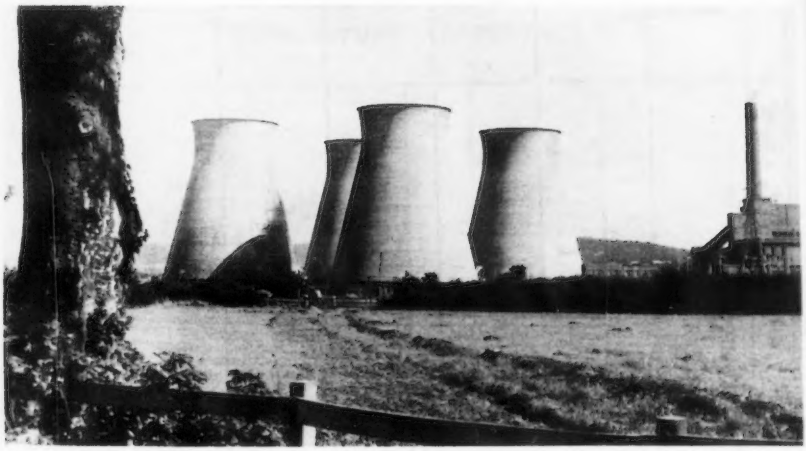


FIG. 17 VIEW OF 300 MW. COOLING TOWER INSTALLATION

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2. Parkinson, G. C., Private Communication.

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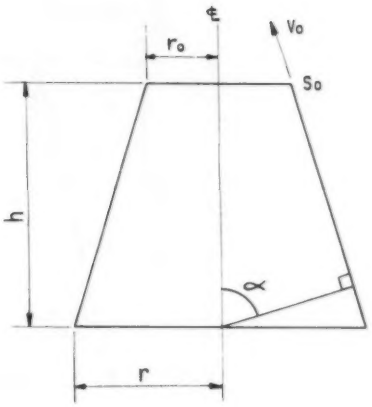
APPENDIX I

ANALYSIS OF "DIABOLO" SHELL

AN EXTENSION OF THE ANALYSIS APPLIED TO THE CYLINDER TO THE MORE COMPLICATED CASE OF THE TRUNCATED CONE WITH A DIRECT FORCE V_o AND SHEAR FORCE S_o ACTING ON THE TOP LEADS TO THE FOLLOWING RELATIONSHIPS:



$$n = r_o/r$$

$$V = -\frac{p''h^2(n+2n^2)}{6r_o \sin^3\alpha} - \frac{p\cos\alpha\, h(1+n)}{2\sin^2\alpha} - \frac{s_o'(n-n^2)}{\cos\alpha} + V_o n$$

$$S = \frac{p'h(1+n+n^2)}{3\sin^2\alpha} + S_o n^2$$

$$t = -pr\,\mathrm{cosec}\,\alpha$$

A SUPERIMPOSED SECTION HAVING A DIFFERENT ANGLE OF INCLINATION α AND WITH DIRECT AND SHEAR FORCES AT ITS BASE V_1 AND S_1 WILL PRODUCE THE FORCES V_o AND S_o AS FOLLOWS :-

V_o = THE COMPONENT OF V_1 IN THE DIRECTION OF THE LOWER SHELL

S_o = S_1 – THE FIRST DIFFERENTIAL WITH RESPECT TO β OF THE COMPONENT OF V_1 IN THE HORIZONTAL DIRECTION.

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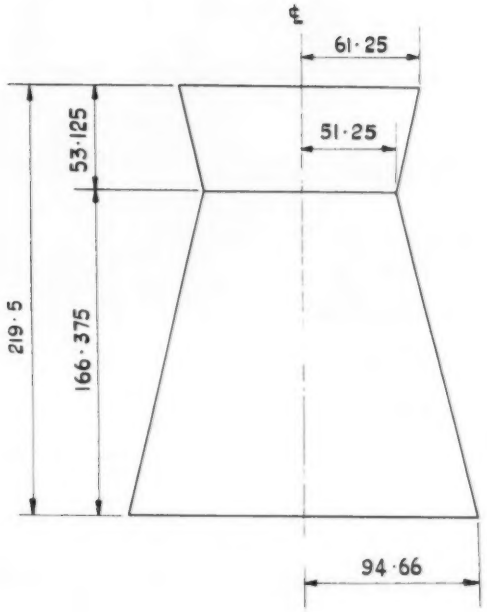
EXAMPLE 2



TOP SECTION

$\alpha = 100° 36'$, $\cos\alpha = -0·184$, $\sin\alpha = 0·983$,

$\sin^2\alpha = 0·966$, $\sin^3\alpha = 0·950$, $h = 53·125$,

$h^2 = 2822$, $n = 1·195$, $n^2 = 1·428$,

$(1+n) = 2·195$, $(1+n+n^2) = 3·623$,

$(n+2n^2) = 4·051$, $r_0 = 61·25$.

$$S = + \frac{p' h (1+n+n^2)}{3 \sin^2\alpha}$$

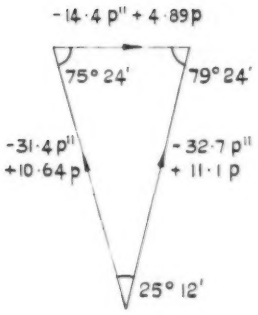
$$= + p' \frac{53·125 \times 3·623}{3 \times 0·966} = \underline{\underline{+ 66·4\, p'}}$$

$$V = - \frac{p'' h^2 (n+2n^2)}{6 r_0 \sin^3\alpha} - \frac{p \cos\alpha\, h (1+n)}{2 \sin^2\alpha}$$

$$= - p'' \frac{2822 \times 4·051}{6 \times 61·25 \times 0·950} + p \frac{0·184 \times 53·125 \times 2·195}{2 \times 0·966}$$

$$= \underline{\underline{- 32·7\, p'' + 11·1\, p}}$$

3



LOWER SECTION

α = 75°24', cos α = 0·252, sin α = 0·968,
$\sin^2\alpha$ = 0·937, $\sin^3\alpha$ = 0·907, h = 166·375,
h^2 = 27580, n = 0·542, n^2 = 0·294,
$(1+n)$ = 1·542, $(1+n+n^2)$ = 1·836, $(n+2n^2)$ = 1·13
$(n-n^2)$ = 0·248, r_0 = 51·25.

$$V_0 = -31{\cdot}4\,p'' + 10{\cdot}64\,p$$

$$S_0 = +66{\cdot}4\,p' - (-14{\cdot}4\,p''' + 4{\cdot}89\,p') = +14\,4\,p''' + 61{\cdot}5\,p'$$

$$S = \frac{p'h(1+n+n^2)}{3\sin^2\alpha} + S_0 n^2 = p'\,\frac{166{\cdot}375 \times 1{\cdot}836}{3 \times 0{\cdot}937} + S_0 n^2$$

$$= 108{\cdot}6\,p' + (18{\cdot}1\,p' + 4{\cdot}23\,p''') = \underline{126{\cdot}7\,p' + 4{\cdot}23\,p'''}$$

$$V = -\frac{p''h^2(n+2n^2)}{6\,r_0\sin^3\alpha} - \frac{p\cos\alpha\,h(1+n)}{2\sin^2\alpha} - \frac{S_0'(n-n^2)}{\cos\alpha} + V_0 n$$

$$= -p''\frac{27580 \times 1{\cdot}13}{6 \times 51{\cdot}25 \times 0{\cdot}907} - p\,\frac{0{\cdot}252 \times 166{\cdot}4 \times 1{\cdot}542}{2 \times 0{\cdot}937} - 0{\cdot}985\,S_0' + 0{\cdot}542\,V_0$$

$$= -111{\cdot}6\,p'' - 34{\cdot}6\,p - (60{\cdot}7\,p'' + 14{\cdot}2\,p^{IV}) + (-17{\cdot}0\,p'' + 5{\cdot}8\,p)$$

$$= \underline{-28{\cdot}8\,p - 189{\cdot}3\,p'' - 14{\cdot}2\,p^{IV}}$$

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Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

AMERICAN POWER-PLANT COOLING TOWERS

J. A. Scarola,[1] A. M. ASCE

ABSTRACT

This paper generally reviews the historical development of American power plant cooling towers and discusses the civil engineering aspects of the presently accepted mechanical draft towers. Some of the problems encountered, the present materials of construction of the tower, and the basin design and construction are outlined in this paper.

INTRODUCTION

Cooling Towers are an important, and in many installations, an indispensable auxiliary of a steam-electric power plant. The great industrial expansion in our country has made prime site locations along rivers and large bodies of water more difficult to find. Extensive studies are currently being conducted to determine the effect, if any, of warm water discharge on fish life in rivers and lakes. Some states are enacting regulations to control pollution and heat rejection into these natural bodies of water. Such enactments may restrict the use of once-through cooling systems in future extensions to existing industrial and utility plants and new installations.

Cooling Towers permit modern power plants to be built closer to load centers where a limited supply of ground or surface water may exist. Although costly in maintenance in the past, the reliability of cooling towers has been proven by the experiences in many installations which depend solely upon a closed system of this type for their circulating water cooling.

This paper generally reviews the development of American power-plant cooling towers and discusses the civil engineering aspects of the construction and installation of the mechanical draft tower.

Note: Discussion open until March 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2228 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 5, October, 1959.

1. Prin. Civ. Engr., Ebasco Services Inc., New York, N. Y.

History

The development of the present day cooling tower is the result of the progressive evolution of the following methods used to cool large quantities of water;

a. Ponds
b. Spray-Type Cooling Ponds
c. Spray-Type Cooling Towers
d. Natural-Draft Cooling Towers
e. Mechanical-Draft Cooling Towers
f. Hyperbolic Natural-Draft Towers

One of the earliest methods of cooling water was to permit the water to flow into an open area or pond. In many installations this cooling method is still utilized, however, large areas are required to expose sufficient water surface for effective cooling. Spray-type cooling ponds followed open ponds. The sprays created additional water surface and reduced land-acreage requirements. However, larger quantities of make-up water were required as the smaller water-droplets were wind-blown from the pond. In order to contain the water-droplets and to protect nearby equipment from damaging effect of this carry-over, louvered fences were installed around the pond; thus the development of the spray-type cooling tower. The louvered walls were carried higher to take advantage of the natural-draft effect of warm air rising through the tower. In addition, horizontal layers of wooden strips, now referred to as packing, were installed within the tower structure to further enlarge the water surface by breakup. These innovations resulted in the development of the natural-draft tower. The natural-draft cooling tower thus was made up of a structure with louvered walls and many layers of wooden packing in the interior. Warm water was discharged into the upper area of the structure and fell through the tower, striking rows of packing and finally collecting in a basin under the tower.

Increasing service requirements called for more reasonable tower sizes. This resulted in the development of mechanical-draft towers which employed motor-driven fans to induce a forced-air circulation of higher velocity. The first mechanical-draft cooling towers were of the forced-draft type with the fans placed at ground level blowing air into the sides of the tower. The entire top of the structure was open to the air-vapor discharge. One of the major drawbacks of this tower was the low exit velocity which permitted moist air to recirculate back into the fan intake, thereby reducing the efficiency of this type of tower design.

Present practice in this country favors induced-draft cooling towers which have the fans placed on the top of the tower. The air-vapor mixture is discharged through a fan stack at the top of the tower. By comparison to the forced-draft tower design, the moisture-laden air is discharged higher into the atmosphere thereby dispersing at a greater distance from the tower. Fig. 1 shows a sketch of a typical forced and an induced-draft cooling tower.

Mechanical-draft cooling towers are further classified as to the air-water flow direction (cross- or counter-flow), and packing design (splash- or film-type). When the air is introduced into the tower below the packing, it is known as a counter-flow tower. In a cross-flow type tower, the air enters the tower at the level of the packing.

In splash-type packing the decks, which continually break up the falling water, are formed approximately 5 to 24 inches apart. In the film-type packing the water flows over the wetted surfaces in a thin continuous film. This film-type packing generally has an egg-crate appearance when viewed from above.

Discharge Vapor and Drift

In large cooling-tower installations careful consideration must be given to tower orientation with respect to prevailing winds to minimize recirculation. Large air flows are required to carry off the discharge vapors and occasionally this moisture-laden air induces fog which may interfere with automobile traffic, railroads or airport activities. In addition, the drift particles discharged from the tower can be very corrosive to adjacent operating equipment. In cold weather the vapor-laden air can form ice deposits on equipment, electric transmission lines and roads. Drift eliminators are provided in all mechanical-draft cooling towers to reduce the carry-over of entrained water particles.

Large structures or natural obstacles in the cooling-tower area should not be overlooked when locating the tower. The cooling tower should be located to permit unobstructed air flow to it and free vapor discharge from it. When locating more than one tower in the area, the recirculation from one tower to the other must be considered. The tower spacing should be sufficiently wide and the towers oriented in a direction to minimize recirculation. A photograph of the vapor discharging from an induced-draft cooling tower, with no visible recirculation, is shown in Fig. 2.

In some instances high-stack cooling towers have been successful in decreasing fogging and recirculation problems by dissipating the vapors into the atmosphere at higher elevations and somewhat greater velocities. These cooling towers may be of the forced- or induced-draft type and some have stacks extending approximately 120 feet above grade.

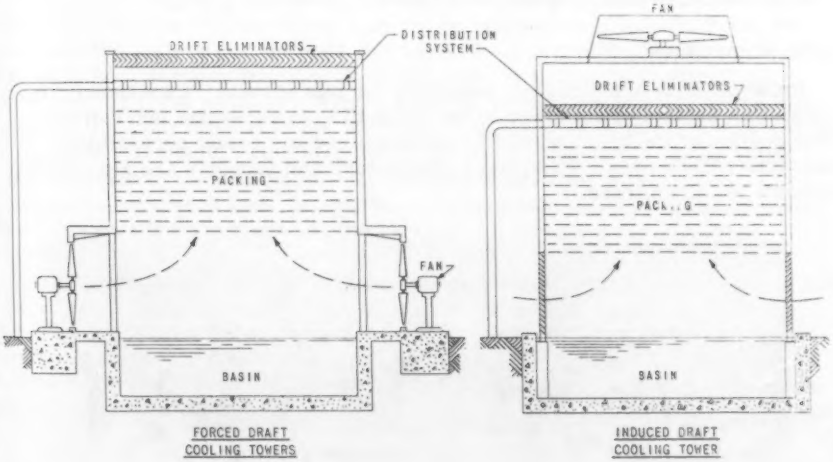

FIGURE 1 - MECHANICAL DRAFT COOLING TOWERS

The use of special material or protective coatings on operating equipment of the tower and adjacent structures may be necessary to minimize the corrosive effects of water treating chemicals deposited by carry-over.

When possible, electrical equipment should be located upwind of the tower and a sufficient distance to decrease the detrimental effect of spray deposits. In some installations, polyvinyl chloride or other special materials have been used for electrical conduit and fittings installed on the tower, while in other cases the conduit has been enclosed in wood troughs for protection. Circulating-water intake pumping equipment and structures installed adjacent to cooling towers usually require more maintenance than if the equipment is placed remote from the tower. Forced-draft towers have the advantage of the fans being placed out of the path of vapor-laden air which usually lowers maintenance on the mechanical and electrical equipment.

Materials of Construction

Experience gained by manufacturers on operating installations has made available various materials of construction. The goal of achieving maximum economy with increased cooling-tower life has opened the field of materials used in towers from primarily redwood construction to asbestos cement, plastics, ceramics, fibreglass and concrete. The following is a brief description of some of the materials used in the various components of the tower.

Fan Stacks

Various fan stack shapes are used to increase fan efficiencies and regain velocity head losses. The material most commonly used is redwood of tongue and groove construction banded together by suitable hardware. Prefabricated sectional wood panels are designed to permit volume production thereby lowering field installation costs. Masonry, tile, metal and fibreglass materials are also used. The excessive corrosion problem at the fan stack has discouraged manufacturers in the use of uncoated ferrous metal.

Drift Eliminators

Redwood slats are generally employed for this service. Experience has shown this section of the tower to be susceptible to excessive deterioration. This portion of the tower should be sectionalized and easily accessible to facilitate replacement. Fiberglass eliminators and supports are also offered by some manufacturers.

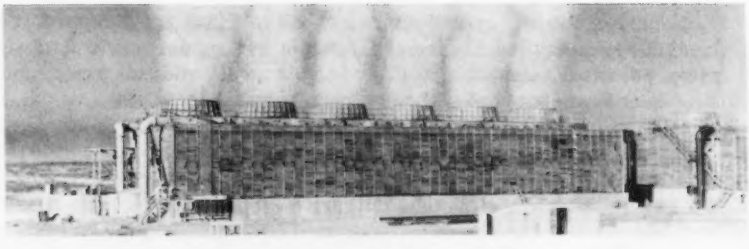

FIGURE 2 - INDUCED DRAFT COOLING TOWER

Packing

The optimum packing design is that which is long life, gives the greatest possible breakup of water for the least material and permits the greatest air flow with the smallest fan horsepower.

Some engineers believe that a heavy packing of at least 1-inch thick lumber should be used in cooling tower construction as the packing is subject to physical erosion as well as chemical attack. Others indicate that with proper water treatment thinner packing can be used. Heartwood Redwood and Douglas Fir lumber untreated and treated, are generally used for this service. Plastic and ceramic packings are also available.

Easy access to the packing for inspection and replacement is very important. The supporting structure for the packing should be carefully designed to prevent sagging or misalignment which could cause water channeling and loss in thermal performance.

Structural Frame

Redwood lumber is normally used for the structural frame. The Cooling Tower Institute has published a set of specifications for this species of lumber in cooling-tower applications. These specifications define the various grades of redwood lumber and specify framework design data.

In conventional towers of multicell construction, wind loads are of major concern. Manufacturers usually supply a standard tower designed for 30 pounds per square foot wind load. A well-planned bracing system that is compatible with the fill arrangement and positive in the distribution of wind shear loads to the foundation is an important requirement. It has been found that some manufacturers divide the longitudinal wind loads, on multicell installations, by removing a portion of the wind load at the windward side and carry the remainder to the leeward side; this redistribution of loading in the design of cooling-tower frames is questionable.

Connection of the tower intermediate columns to the longitudinal struts cause them to act as a partially restrained connection, thereby permitting each column to carry a portion of the wind shear load to the basin floor. Although the interior column bases may not be fastened to the basin floor, the dead and live load on the columns will allow them to transmit wind shear to the basin floor through friction. It is therefore possible, in multicell construction, for these wind forces to be carried through the bracing to the foundation in the cells closest to the windward end. Some design engineers are of the opinion that the full wind load should be considered acting on the entire windward face of the structure, with the load distributed within the first two cells on the windward end. In addition, no more than possibly three consecutive cells should be unbraced in the longitudinal direction for increased stability.

Earthquake, snow loadings or other special considerations are incorporated in the frame designs by tower manufacturers when requested. Earthquake and wind loadings are usually not considered as acting simultaneously. For members subject to wind or earthquake loading, the allowable stresses are normally increased by 33-1/3% over the normal allowable working stresses. No allowable increase in stress should be permitted for snow-load conditions.

Casing

Double walled construction of either redwood or cement-asbestos board is generally used. The primary purpose of the outer wall is to cover what might be an unsightly appearance of discoloration caused by water leakage through the inner wall. It is important to properly ventilate the space between the walls to prevent decay. The casing is sometimes factory preassembled to take advantage of more economical construction. Masonry enclosures are also offered by some manufacturers.

Experience has shown that the negative pressure or suction at the leeward side of the tower should be carefully considered in the design of the casing. Some designers are of the opinion that approximately 20 pounds per square foot negative load should be used.

Fig. 3 shows the erection of the framing and casing on a partially completed cooling tower.

Water Distribution System

Designs vary widely in type of materials used for the water distributing system. Internal or external piping arrangements are available consisting of steel, cast iron or woodstave pipe. The nozzles and distributing orifices should be of corrosion-resistant materials.

In order to gain full effectiveness of the tower, water distribution should be uniform. There are many means of distributing water over the tower, such as spray pipes, troughs or pools. Control of water distribution to individual cells is usually accomplished by gates, valves, orifices or weirs.

Periodic maintenance of the distributing system is very important as the orifices or nozzles are subject to plugging from debris blown into the tower and algae growth.

Mechanical Equipment and Support

The mechanical equipment is subjected to strenuous duty and should be able to withstand severe corrosion. Present day fans are made up to 22 feet in diameter which employ 125 horsepower motors. Aluminum, stainless steel and monel are commonly used fan materials.

FIGURE 3 - VIEW OF PARTIALLY COMPLETED TOWER

The fan drive support is usually fabricated from structural steel and coated with some form of corrosion protection. Reinforced concrete is also used for the support of this machinery.

The supports for these large fans should be carefully designed to prevent shaft misalignment and to keep vibration to a minimum.

Frame Connectors and Hardware

Experience and service application are an important consideration in the selection of the hardware and frame connector material. Bronze or galvanized steel are commonly used. Caution should be exercised to insure that the various materials used for connectors and hardware are compatible to avoid electrolytic deterioration.

Wood Treatment

The treatment of wood for cooling-tower service continues to be a controversial subject. Those in favor of preservative treatment feel that less costly species of treated timber could be utilized in place of redwood; also that treated wood has withstood the test of time in some applications. Others indicate that the treating compounds are leached out of wood by the cooling water. Wood preservatives that have been used for this service are Creosote, Erdalith, Celcure, Chemonite and others. The climate in a cooling tower is very favorable to fungus deterioration. This biological attack is more noticeable in the drift eliminators and the fan stack zone, and most engineers agree that some treatment is necessary in these areas.

Chemical attack may take place when the lumber is intermittently or continually wetted. Many feel this type attack can be controlled by proper water treating practices and frequent inspection.

Basin Design and Construction

The basin area should be large enough to collect all the water falling through the tower. Sufficient volume of water should be provided in the basin to meet emergency situations that may arise. A break in the circulating-water conduit or water-distributing system of the tower can exhaust the water supply in the basin in a short time. Therefore, a quantity of water which will provide at least 5 minutes of operation, after such an emergency occurs, will allow operators enough time to shut down equipment before damage can occur. Basins, with at least this much storage, will offer additional operating reliability as there will be enough water for several hours of operation in case of failure of the make-up water system.

Other factors to be considered in sizing the basin are the hydraulic losses and the effects of the operating water level in the basin on the circulating-water pump submergence. In some installations, the basins are sectionalized for cleaning purposes, therefore water volume in the various sections should be carefully sized for the required operating schedule when being cleaned.

Provision should be made in the basin to incorporate a make-up line, a blowdown line and an overflow line or weir. The make-up system is required to keep the basin supplied with sufficient water to compensate for losses due to evaporation and drift. This is an important element in a cooling-tower

installation and special care should be exercised in order to provide a reliable make-up system. The blowdown, which is the wasting of a small quantity of circulating water, is necessary to control the concentration of chemicals in the water. The overflow line or weir is usually directed to an area which will carry off the excess water which has risen above a predetermined level. A low-level alarm should be installed in the basin to indicate insufficient or loss of make-up supply.

The large body of available water in the basin is sometimes considered as a portion of the fire-fighting system in power plants. The water in the basin can be used as a stand-by to supplement the fire system in case of an emergency, however, in very few installations, if any, is it the sole source of water storage for fire fighting. In colder climates, the cooling-tower basin of intermittent duty cooling towers must be drained to prevent damage caused by freezing. In other locations, debris may be such that more frequent shutdowns are required to facilitate cleaning.

The foundation is an important factor in the performance of the tower and certain precautions should always be undertaken to attain the proper design and construction.

In mechanical draft timber towers with concrete basins, the soil load is generally in the vicinity of 1200 pounds per square foot. At most locations soil loads of this magnitude will not create any difficulties. However, if the nature of the soil is such that unequal settlements may occur, means must be provided to prevent or compensate for the expected amount of settlement. Periodic level readings and inspection with necessary adjustment of the supporting structure should be instituted if expected settlement is considered to be within practical limitation for adjustment. However, if the magnitude of expected settlement is beyond this scope then preloading, densifying the subgrade, or the installation of piles should be used for satisfactory support.

Excessive unequal settlement can cause cracks in the basin walls resulting in water leakage. The tower structure can be seriously weakened by unequal settlement as distorted structural members will not distribute their loads properly, thereby overstressing other members and causing failure.

Material

Large tower basins are usually constructed of concrete, however, basins of wood, steel and earth-lined construction have also been employed successfully, depending upon the geographical location and size of tower.

When concrete construction is used for the basin, the mix should be designed and controlled to attain maximum density and low permeability. In this manner, maximum water tightness of the basin walls and slab may be obtained. The bottom of the basin should be carried to a depth below the frost line. Design practice has been to use a concrete mix which will result in a 28-day design strength of 3000 pounds per square inch. Sufficient reinforcing steel should be used to meet design conditions. Reinforcing steel should be placed so as to have a minimum of 2 inches of concrete cover over the steel.

The basin usually consists of a continuous flat slab with construction joints alternating with expansion joints at approximately 30-foot intervals. These joints will facilitate construction and allow for controlled expansion or contraction. Water stops of metal, rubber, or plastic should be used at all joints and made continuous, with all splices and corner connections properly jointed. All expansion joints should be filled with a resilient-type filler material. In

large installations, the basin slab is normally about 1 foot thick and sloped to a sump or low point to facilitate draining and cleaning when necessary.

The walls are normally 5 to 6 feet in height and approximately 1 foot thick. The wall is usually of sufficient height to include 9 to 12 inches of freeboard above the maximum operating water level. Construction and expansion joints should also be provided in the walls. A sketch of a concrete basin for a timber-type mechanical-draft cooling tower is shown in Fig. 4 and a photograph is shown in Fig. 5.

The concrete basin design for the ceramic-type tower is considerably more extensive than for timber towers because of the weight of the materials used in the tower construction. Unequal settlement of the basin is more of a concern with this type tower as experience in other applications has shown that an improper foundation supporting tile or block construction may create a very poor appearance by cracking. Expansion and construction joints are also an important requirement for these basins.

Other types of material and construction practices have been considered for tower basins. The structural members of the towers can be supported by individual concrete footings properly designed for the soil conditions encountered. The water containing basin would be then formed in an earth enclosure lined with asphalt or gunite. The lining may consist of poured-in-place asphalt or asphalt sheets. Asphalt sheets have recently been gaining wide acceptance for this type of construction. The subgrade should be well compacted and rolled, after which a weed killer is applied to the prepared subgrade before placing the lining. All column footings should be treated with an asphalt primer before placing the asphalt. This primer application should be carefully placed against the footings to secure a firm bond to reduce leakage at these points.

All anchor bolts or inserts should be accurately set in the concrete to minimize field adjustment of framing for proper fit and bearing. Selection of the type of material used for these anchors is of the utmost importance as the corrosion action on the exposed portion of the anchor bolts has been very severe in some instances. This can create a costly and annoying problem of replacement.

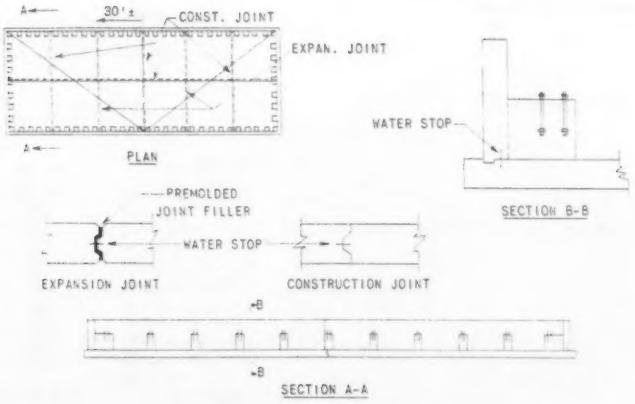

FIGURE 4 - COOLING TOWER CONCRETE BASIN

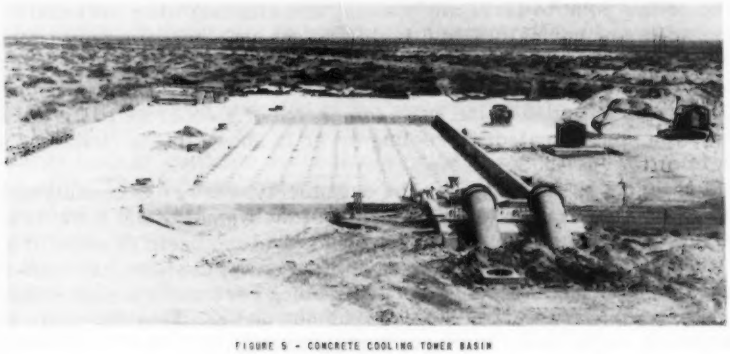

FIGURE 5 - CONCRETE COOLING TOWER BASIN

The available materials, construction or maintenance practice in different areas will usually dictate the economics of the type of design to be used in the tower basin.

Outlet and Pump Locations

The basin outlets are usually provided with a system of screens to keep leaves and debris out of the circulating-water system. Normally, the screens are kept clean by periodic hand-raking and the debris is carried from the deck of the outlet structure. A means of isolating the tower from the circulating-water conduit is included in this structure in case unwatering of either is required. Slide gates or stop logs are normally used for this purpose. Fig. 6 shows typical details that have been used for stop logs in some installations.

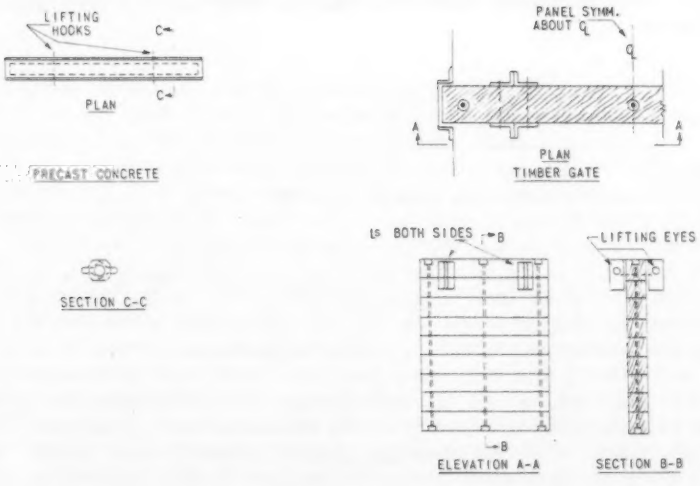

FIGURE 6 - TYPICAL STOP LOGS

The location of the circulating-water pumps, for respective installations, depends on factors such as tower arrangement, type plant (basement or grade), type pumps (vertical or horizontal), pumping costs and circulating-water conduit economics. Evaluation of the above factors has resulted in pump locations either at the cooling tower or at a remote distance from the tower. Fig. 7 shows a sketch of a typical basin intake structure when circulating-water pumps are remote from the tower. Fig. 8 shows a sketch of an intake structure with circulating-water pumps adjacent to the cooling-tower basin.

Intakes have been placed adjacent to the towers at the ends, or on the long sides of the basin. If the pumps are placed in the intake chamber adjacent to the tower, care should be exercised in the design of the transition from the tower basin to the pump chamber. For the combined pump and intake chamber design, a greater depth of structure is usually required to give sufficient submergence for the circulating-water pumps. In order to keep turbulence at a minimum and obtain a uniform water flow at the pump chamber entrance, low velocities should be maintained. In addition, baffles may be required to effect proper distribution of water to the pump chamber. It may also be necessary to provide a means for directing the flow to the tunnel or pump chamber when part of the tower is shut down. Fig. 9 is a photograph of a cooling tower and adjacent pump chamber.

Fire Precautions

It is important that fire safety measures be taken during the construction of timber cooling towers. Combustible waste materials should be removed immediately and "No Smoking" regulations enforced. Strict control of welding operations or flame cutting is also necessary.

Some plant operators are of the opinion that an operating tower is inherently protected as its many portions are flooded. However, it is possible for

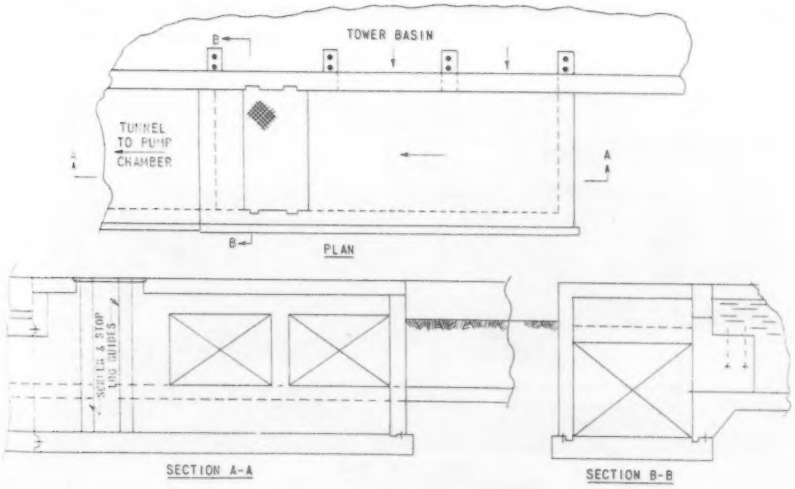

FIGURE 7 - COOLING TOWER SCREEN CHAMBER

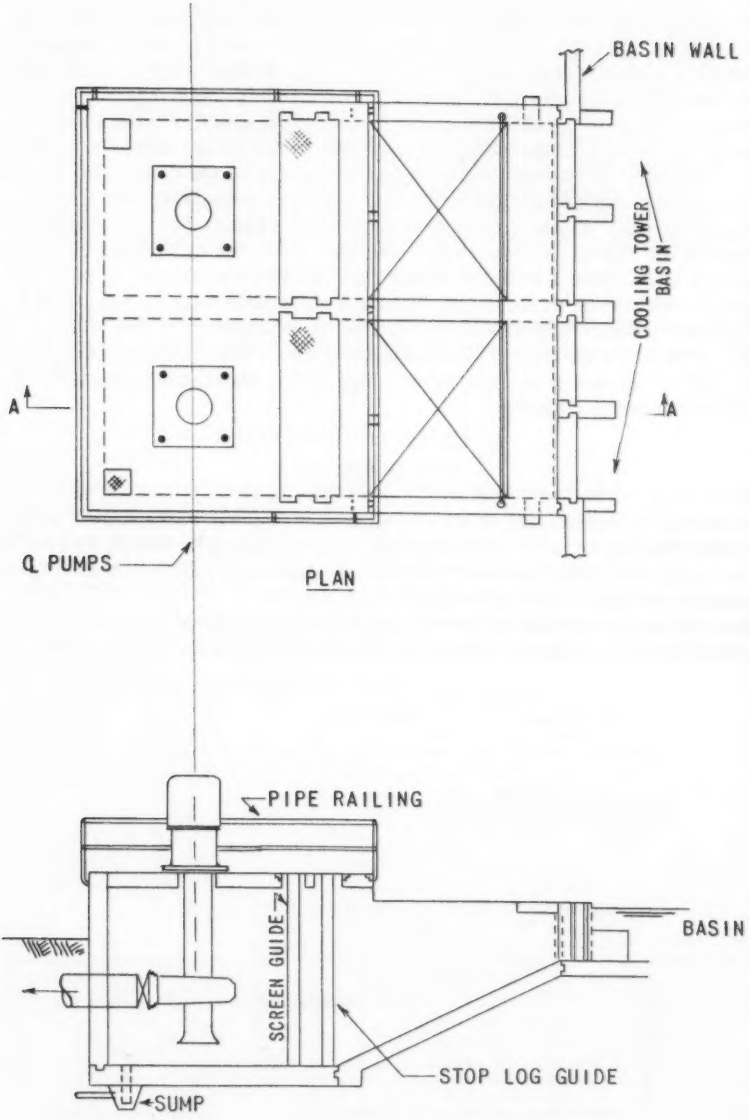

FIGURE 8 -PUMP & SCREEN CHAMBER

strong winds to dry out the windward sheathing even though the tower is operating, therefore care should be exercised at all times. During shutdowns, it is necessary to take extreme precautions against fire as most of the tower is dry.

Ground areas around the towers should be kept clear of weeds, brush or other easily ignited materials.

Some type of hydrant or sprinkler system should be installed around the conventional wood towers.

Trends

Cooling tower manufacturers are continually attempting to develop a more lasting product within economical boundaries. High maintenance costs, with the earlier design towers, has resulted in a present trend toward more permanent materials of construction. However, not all of these materials have yet reached the competitive price stage. Future improvements in design, fabrication and construction methods should help remedy this situation.

Hyperbolic Cooling Towers

Considerable interest is presently being expressed in this country, in the European design of the natural-draft hyperbolic cooling tower. There is much to be said in favor of this long-life all-concrete structure. Detailed design studies and cost investigations have been made of this tower recently by several tower manufacturers for installations in this country. Although these concrete towers have a higher initial cost than the conventional all-redwood towers, recent studies show their over-all economics to appear attractive in certain situations. They reportedly possess the advantage of complete vapor dispersal aboveground, lack of fan noise and no recirculation. One may well predict that we will soon see some natural-draft hyperbolic cooling tower installations in this country.

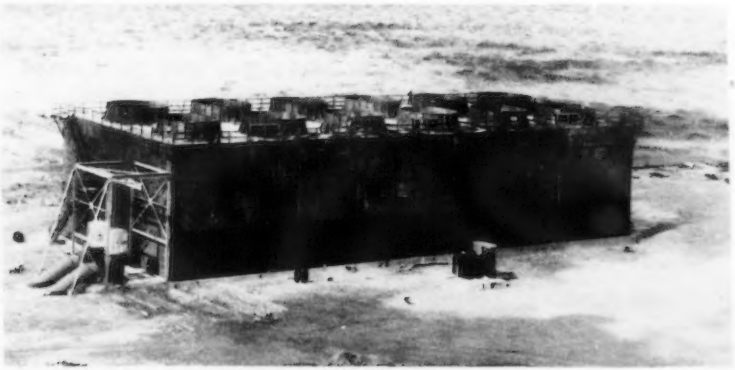

FIGURE 9 - COOLING TOWER AND PUMP CHAMBER

ACKNOWLEDGMENT

The author wishes to express his appreciation to Mr. M. W. Larinoff, Mechanical Engineer, Ebasco Services Incorporated for his help in preparation of this paper.

He also wishes to thank his associates in design and construction for their assistance.

In addition, appreciation is due several of the leading Cooling Tower manufacturers for their cooperation in supplying construction data.

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MONTGOMERY DAM - ROCK FILL WITH ASPHALTIC CONCRETE DECK[a]

Discussion by G. R. Scott and J. L. Sherard

G. R. SCOTT,[1] F. ASCE.—The authors' comprehensive description of the development of this project leaves nothing of importance to add relating to the design of the structure. This discussion is submitted primarily to record the behavior of the dam with regards to settlement and horizontal movements after having been filled two times and to describe briefly, the effect on the asphaltic concrete membrane of one complete cycle of filling and emptying of the reservoir.

The rock fill was completed in October 1956, the asphaltic concrete membrane was placed during the summer of 1957 and the reservoir was filled in 1958. It was emptied during the winter and was filled again by early July in 1959.

Bench marks were established on the crest of the dam after the completion of the rock fill in October, 1956 and levels were run again in June of the following year to measure the amount of settlement during the winter. These bench marks were destroyed during the installation of the asphaltic concrete membrane in 1957, but others were established at the conclusion of the work from which subsequent measurements have been made. These final bench marks were not at the same locations as the earlier ones and the settlement values of June 10, 1957 shown in the tabulation have been adjusted from the original measurements to conform with the stations indicated.

The horizontal movements have been measured from a series of reference points on the tops of the steel posts supporting the upstream parapet. Obviously such points are not ideal for the purpose of measuring the true horizontal movement of the dam as they will reflect as well the amount of any tilting experienced by the post. However, the fact that the measured displacements have been small and that the upstream wall is not distorted is evidence of very little horizontal movement of the dam itself.

The bench marks are designated by station numbers along the axis of the dam. Station 5+00 is at the extreme south end of the dam. The bench mark at 7+95 is at a point where the fill is 62 feet deep; the point of maximum fill (113 feet) is about station 13+00 and the depth of fill at station 21+45 is 35 feet. The depths of fill at the other bench marks may be approximated by assuming uniform slopes between the end bench marks and station 13+00.

The results of the measurements are given in the two following tabulations. The measurements made in 1958 and 1959 were made at practically full reservoir:

a. Proc. Paper 1556, February, 1958, F. W. Scheidenhelm.
1. Black & Veatch Cons. Engrs., Kansas City, Mo.

Montgomery Dam

Settlement Data

Station	From Oct. 1, 1956 To June 10, 1957	From Sept. 24, 1957 To Aug. 20, 1958	From Aug. 20, 1958 To July 1, 1959
7+95	0.03 ft	0.06	0.06
9+45	.04	0.12	0.04
10+95	.05	0.15	0.05
12+45	.05	0.16	0.07
13+95	.05	0.18	0.03
15+45	.05	0.16	0.04
16+95	.04	0.10	0.04
18+45	.05	0.03	0.01
19+95	.04	0.00	-0.01(a)
21+45	.04	0.01	0.00

(a) Represents a raising above the preceding year.

Montgomery Dam

Horizontal Movement of Crest

Station	From Sept. 24, 1957 To Aug. 20, 1958		From Sept. 24, 1957 To July 1, 1959	
7+95	0.10 ft	U	0.08 ft	U
9+45	0.10	U	0.08	U
10+95	0.02	U	0.16	U
12+45	0.02	D	0.06	U
13+95	0.00	-	0.07	U
15+45	0.00	-	0.06	U
16+95	0.06	U	0.05	U
18+45	0.00	-	0.08	U
19+95	0.05	D	0.11	U
21+45	0.00	-	0.05	U

U = Upstream movement

D = Downstream movement

A careful examination was made of the surface of the asphaltic concrete in 1959 before filling for the purpose of estimating the extent (if any) of damage due to ice and to locate and repair any cracks which might have opened due to settlement or contraction.

Apparently, the effect of ice on the surface of the pavement was negligible because the surface sheen caused by the compacting roller appeared fresh and at no place was it noticed that any of the surface had been pulled loose by adherence to the ice as it must have broken away and dropped with the water level during the winter.

Two short cracks were noticed near the north end of the dam. One was a horizontal crack located above the water line, and the other was a vertical crack near the top which followed a construction joint. Both joints were cleaned out and filled with material used on highways in the area. The asphalt binder had a penetration at 77 F of 120-300, fluxed or blended with a kerosene type solvent. The aggregate was well graded with 100 percent passing 3/4 inch mesh. The mix used for repair purposes contained from 5 to 7 percent of asphalt binder.

J. L. SHERARD,[1] M. ASCE.—The authors have performed a valuable service in summarizing for the profession the problems which they encountered in the design and construction of Montgomery Dam. The use of a 12 inch thick asphaltic concrete upstream slab as a water barrier on a rockfill dam of this size is a considerable break with past practice.

The writer's first reaction when hearing several years ago that this dam was to be constructed to an eventual height of over 150 feet with an asphalt upstream face in the high Rocky Mountains was incredulity. This has changed gradually to respect for the foresight and the courage demonstrated by the engineers who have successfully completed this very interesting structure.

As a result of their investigation, the authors concluded "From the evidence available, there is good reason to believe that asphaltic concrete, when properly designed, placed and compacted, will be at least as durable as competitive materials for such deck construction." While many engineers will be skeptical of this statement, their skepticism would be lessened appreciably if they had the opportunity to examine personally a sample of the compacted asphaltic concrete used at Montgomery Dam. The writer had an opportunity to examine a 4 inch diameter core which had been cut from the compacted asphaltic concrete probably from one of the test sections. This suffered little by comparison with ordinary Portland cement concrete in density, hardness and apparent durability.

In their comparison between decks of asphaltic and Portland cement concrete, the authors discuss the fact that a upstream zone of hand-placed or crane-placed rock would probably have been used under a concrete slab. The writer can see no necessity for preparing a better base for a slab of reinforced concrete than would be required under the asphaltic concrete slab used.

One of the features of the project which probably had an influence on the decision to use asphaltic concrete, in spite of the fact that there is no direct precedent for it, was the fact that the dam retained a relatively small reservoir which would be emptied each year during normal operation. Also with a 6 foot diameter outlet conduit, it is possible to lower the reservoir level fairly rapidly in the event that an unexpectedly large leak developed. From

1. Woodward-Clyde-Sherard and Associates, New York, N. Y.

this standpoint, it seems that the situation was favorable for the trial of the asphaltic concrete.

The authors point out that the 1.7:1 upstream slope used was primarily governed by the properties and construction problems of the asphaltic concrete deck. Laboratory "slope-flow" tests led to the conclusion that the asphaltic concrete could be placed on slopes as steep as 1.5:1 and would remain stable even if the temperature rose to 140 degrees Fahrenheit. It would be very interesting if the authors would describe briefly how these "slope-flow" tests were performed.

It would be interesting if the authors would elaborate in their closing discussion on their thoughts regarding the influence which ice on the surface of the reservoir may have on the performance of the asphaltic concrete deck. How thick an ice sheet is expected? Will this ice sheet bond securely to the upstream deck? If so, what might happen if the reservoir level lowered removing the support while the ice sheet was still thick? The writer has seen relatively large boulders torn out of rock riprap blankets by such action.

Even though the laboratory tests indicated the asphaltic concrete would be stable on 1.5:1 slope, the upstream slope was chosen as 1.7:1 to provide a little added factor of safety and to facilitate construction. In the comparison between the costs of upstream decks from Portland cement and asphaltic concrete, computations indicated that the cost of the asphaltic concrete was approximately 50% of that of the Portland cement concrete. If Portland cement concrete had been used, the upstream slope for a dam of this height could have been constructed considerably steeper with a resulting cost advantage in favor of the Portland cement concrete slab.

In order to assure that no hydrostatic pressures would develop behind the asphaltic concrete deck when the reservoir was being lowered, the rock directly beneath the deck (Zone 2) was screened to remove all rocks smaller than 3 inches. As a leveling course and a base for the deck, a layer of finer rock graded between 3/4 inches and 3 inches was used. Fig. 7 shows a vibratory roller compacting this surfacing material. It would be valuable if the authors would describe the average and maximum thickness of this finer material used directly under the asphaltic concrete deck.

As stated in the paper, one of the principal advantages of the asphaltic concrete deck was the fact that it could be constructed much more rapidly than a Portland cement concrete slab. It should be pointed out that an upstream face of steel plate can also be placed very rapidly. During 1958, a rockfill dam, approximately 80 feet high, was constructed by the City of Caracas, Venezuela, for an expansion of their municipal water storage. One of the principal requirements imposed upon the designers of this dam was the fact that it had to be both designed primarily because it could be erected rapidly in wet weather. While its initial cost is higher, the steel plate decking is probably more flexible and better able to follow differential settlements than either asphaltic or Portland cement concrete and also probably has better resistance against the erosive action of waves than asphaltic concrete.

More than a year has elapsed since the authors prepared the paper. It is hoped that in their closing discussion they will be able to give some additional information regarding the performance of the dam. Also in the closing paragraph of the paper, it was stated that a hydro-electric project including a dam similar to Montgomery Dam but twice as high was scheduled for construction in 1958. Further details of this dam would be particularly welcomed by all dam engineers.

ROCKFILL DAMS: KENNEY & CHEAKAMUS[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] F. ASCE.—The author has given an unusually interesting comparison in his treatment of the Kenney and Cheakamus dams, both of the rockfill type but varying materially in design of cross-section and especially in foundation conditions, Kenney dam being founded on rock and Cheakamus dam on a 100-year-old landslide. This discussion is confined to the performance of the Kenney dam since its construction.

The author's Fig. 2 illustrates the rate at which hydrostatic loading on Kenney dam was imposed. Reservoir filling began October 8, 1952, with full reservoir at el. 2800 being reached August 1957.

As stated by the author, no seepage has been detected coming from the abutments or through the embankment proper, although the possibility of leakage through the abutments did exist, because of the contacts between successive lava flows beyond the limits of the foundation grouting.

During construction, two sets of settlement (and deflection) movement plates were installed. One of these sets was located in the top of the impervious core about 5 ft. below the crest, which consists of rockfill with a gravel topping arranged in the manner shown by the author's Fig. 1. The second set was located about 3 ft. below the surface of the berm at el. 2785, in the top of the sluiced rockfill. The first set comprised 13 monuments spaced at 100 feet intervals along the base line and on a vertical camber or curve of 3.5 feet middle ordinate. The second series of 11 monuments in the berm at el. 2785.0 was initially in a horizontal plane at el. 2782.75 or about 36 ft. below those in the crest.

Location and other details of the settlement plates in the crest of the dam are given by Table 1, with corresponding data for those in the berm by Table 2. Settlement observations for the crest and the berm have been made on seven occasions, with the last in July, 1958. Data is given by Tables 3 and 4.

It will be noted:

1. The present settlement of both the crest and the berm monuments has been relatively uniform, percentage-wise. That is, the settlement is generally proportional to the height of the rockfill, at the monument.
2. Average berm settlement is about 42% greater than the average settlement of the crest.
3. Settlement of the crest has slowed down and appears to be virtually complete. See Fig. 1.

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a. Proc. Paper 1671, June, 1958, by W. G. Huber.

1. Vice-Pres. and Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

4. Settlement of the berm has slowed down but does not appear to have reached the same degree of completion as that for the crest. See Fig. 2.

At the same time as settlement observations were made, measurements were made to determine the deflection movement of the monuments in a downstream direction, Table 5 giving essential details for the crest and Table 6 corresponding data for the berm. The great majority of the movements are in the downstream direction but a few measurements indicate an upstream movement. There appears to be some correlation between deflection and height of fill. The deflection expressed as a percentage of height of fill does not vary greatly, averaging about 0.36% for the crest and 0.47% for the berm. These fingers represent 67% and 61%, respectively, of the corresponding average settlements. The deflections appear to have reached a stable condition, with little if any further increase likely. In fact, there is an apparent decrease between July, 1957, and July, 1958, observations.

TABLE I
KENNEY DAM
SETTLEMENT DATA - CREST

Settlement Plate No.	Location at chainage	Elevation of settlement plate Dec. 12-13, 1952	Elevation of foundation rock	Original height of settlement plate above foundation rock ft.
1	33 + 50	2816.34	2766.0	50.3
2	34 + 50	2817.42	2731.0	86.4
3	35 + 50	2817.89	2691.0	126.9
4	36 + 50	2818.94	2652.0	166.9
5	37 + 50	2819.43	2626.0	193.4
6	38 + 50	2819.69	2552.0	267.7
7	39 + 50	2819.82	2510.5	309.3
8	40 + 50	2819.73	2575.0	244.7
9	41 + 50	2819.44	2607.5	211.9
10	42 + 50	2816.96	2646.0	171.0
11	43 + 50	2818.26	2670.0	148.3
12	44 + 50	2817.40	2698.0	119.4
13	45 + 50	2816.30	2744.5	71.8

TABLE II
KENNEY DAM
SETTLEMENT DATA - BERM AT EL. 2785

Settlement Plate No.	Location at chainage	Elevation of settlement plate on Dec. 12-13,1952	Elevation of foundation rock	Height of settlement plate above foundation rock.ft.
1	34 + 50	2782.82	2731.0	51.8
2	35 + 50	2782.79	2691.0	91.8
3	36 + 50	2782.76	2652.0	130.8
4	37 + 50	2782.71	2626.0	156.7
5	38 + 50	2782.69	2552.0	230.7
6	39 + 50	2782.70	2510.5	272.2
7	40 + 50	2782.70	2575.0	207.7
8	41 + 50	2782.70	2607.5	175.2
9	42 + 50	2782.78	2646.0	136.8
10	43 + 50	2782.78	2670.0	112.8
11	44 + 50	2786.95	2698.0	89.0

Observations have also been made to determine whether there has been any movement of the fill in a direction parallel to the axis of the dam, using the same monuments as for settlement and deflection. Table 7 indicates measurements of longitudinal movement for the crest and Table 8 corresponding movements for the berm. Pertinent comments are:

1. There is no apparent correlation between longitudinal movement and height of rockfill for either the crest or the berm.
2. Actual movements appear to be a maximum near the right abutment and a minimum near the centre. That is, these movements appear to be at a maximum where there is a minimum height of rockfill and a minimum where there is a maximum height of rockfill. This appears to be logical because the movement, while caused by a resultant of several forces, is undoubtedly influenced by the cross section of the valley at the site and possibly by the procedure followed in placement of fill. In

TABLE III
KENNEY DAM
SETTLEMENT OBSERVATIONS - CREST, IN FT.

Settlement Plate No.	Dec.12-13 1952	June 17-22 1953	Nov.20-23 1953	July 17 1954	Sept. 1955	June 1956	July 19 1957	July 1958	Settlement to July,1958, as percentage of height of fill
1	0	0.07	0.09	0.11	0.09	0.02	0.18	0.00	0
2	0	0.19	0.22	0.33	0.35	0.42	0.48	0.43	0.50
3	0	0.29	0.51	0.58	0.63	0.66	0.83	0.83	0.65
4	0	0.42	0.73	0.83	0.90	0.82	0.74	0.90	0.54
5	0	0.77	0.92	1.03	1.16	1.32	1.46	1.57	0.81
6	0	0.84	1.07	1.20	1.33	1.51	1.65	0.68	0.63
7	0	0.59	1.10	1.23	1.35	1.50	1.67	1.73	0.56
8	0	0.58	1.05	1.18	1.32	1.43	1.60	1.63	0.67
9	0	0.46	0.86	0.98	1.07	1.13	1.33	1.29	0.61
10	0	damaged	damaged	damaged	damaged	damaged	damaged	damaged	-
11	0	0.31	0.55	0.62	0.67	0.83	0.90	0.89	0.60
12	0	0.21	0.33	0.38	0.39	0.49	0.57	0.53	0.44
13	0	0.15	0.19	0.23	0.22	0.23	0.36	0.30	0.42
Average settlement									0.54

TABLE IV
KENNEY DAM
SETTLEMENT OBSERVATIONS - BERM AT EL. 2785, IN FT.

Settlement Plate No.	Dec.12-13 1952	June 17-22 1953	Nov.20-23 1953	July 17 1954	Sept. 1955	June 1956	July 19 1957	July	Settlement to July,1958 as percentage of height of fill
1	0	0.12	0.20	0.25	0.27	0.31	0.36	0.26	0.50
2	0	0.24	0.47	0.55	0.60	0.63	0.73	0.80	0.87
3	0	0.32	0.66	0.76	0.83	0.89	1.07	1.06	0.81
4	0	0.43	0.88	1.00	1.09	1.14	1.31	1.31	0.84
5	0	0.55	1.07	1.22	1.33	1.40	1.61	1.61	0.70
6	0	0.64	1.12	1.26	1.37	1.53	1.66	1.61	0.59
7	0	0.47	1.01	1.13	1.24	1.31	1.44	1.49	0.72
8	0	0.40	0.84	0.96	1.05	1.08	1.24	1.22	0.70
9	0	0.33	0.70	0.77	0.85	0.90	1.07	1.12	0.82
10	0	0.30	0.53	0.61	0.67	0.72	0.79	1.00	0.89
11	0	0.18	0.30	0.36	0.39	0.41	0.53	0.96	1.08
Average settlement									0.77

the case of Kenney dam, with the exception of the upper portion, "---- the rockfill was built out from the left abutment in lifts approximately 40 ft. high, with each load sluiced down the slope from the top of the lift by skid mounted monitors delivering about two parts of water to one part of rock." It would appear logical any longitudinal movement in the fill should reveal itself to the maximum extent near the right abutment.

3. The movements appear to have attained a maximum by July, 1957, as would be anticipated due to the hydrostatic loading having reached a maximum that year.

The settlement, deflection and longitudinal movements given in Tables 3 to 8 are relatively erratic, those though possessing broad over-all consistency, as as would be expected from consideration of the behavior of the fill materials.

Generally speaking, Kenney dam appears to have reacted to the imposition of full hydrostatic loading in the same manner as other well-constructed

TABLE V
KENNEY DAM
DOWNSTREAM DEFLECTION - CREST, IN FT.

Settlement Plate No.	Dec.12-13 1952	June 17-22 1953	Nov. 20 1953	July 15-17 1954	Sept. 1955	June 1956	July 19 1957	July 1958	Downstream deflection as a percentage of height of fill
1	0	0.01	0.06	0.02	0.03	0.04	0.10	0.19	0.38
2	0	-0.03	0.16	0.18	0.20	0.12	0.33	0.15	0.17
3	0	0.18	0.36	0.33	0.41	0.49	0.55	0.58	0.46
4	0	0.25	0.43	0.44	0.65	0.62	N.A.	0.71	-
5	0	0.39	0.58	0.72	0.85	0.83	1.08	0.98	0.56
6	0	0.36	0.81	0.79	0.91	0.90	1.16	-	-
7	0	0.49	0.85	0.83	0.98	0.93	1.23	1.04	0.34
8	0	0.39	0.69	0.73	0.90	0.97	1.11	0.76	0.31
9	0	0.30	0.62	0.62	0.76	0.81	0.95	0.60	0.28
10	Damaged	Damaged	Damaged	Damaged	Damaged	Damaged	Damaged	Damaged	-
11	0	0.24	0.41	0.34	0.43	0.57	0.56	0.66	0.45
12	0	0.10	0.31	0.19	0.36	0.37	0.32	0.30	0.25
13	0	0.01	0.20	0.11	0.12	0.16	0.15	0.26	0.36
Average deflection									0.36

Note:- (-) denotes upstream deflection

TABLE VI
KENNEY DAM
DOWNSTREAM DEFLECTION - BERM AT EL. 2785, IN FT

Settlement Plate No.	Dec.12-13 1952	June 17-22 1953	Nov. 20 1953	July 15-17 1954	Sept. 1955	June 1956	July 19 1957	July 19 1958	Downstream deflection as a percentage of height of fill
1	0	-0.27	-0.49	-0.28	-0.34	0.28	-0.27	-0.30	-0.58
2	0	0.17	0.22	0.86	0.33	0.31	0.47	0.38	0.41
3	0	0.15	0.35	0.36	0.50	0.56	0.64	0.63	0.48
4	0	0.21	0.55	0.68	0.72	0.77	0.84	0.88	0.56
5	0	0.39	0.67	0.75	0.88	0.95	0.99	-	-
6	0	0.27	0.55	0.56	1.38	1.48	0.91	-	-
7	0	0.22	0.63	0.55	0.73	0.78	0.90	1.03	0.50
8	0	0.28	0.57	0.54	0.77	0.81	0.83	0.98	0.56
9	0	0.22	0.46	0.48	0.40	0.43	0.73	0.54	0.39
10	0	0.18	0.38	0.38	0.50	0.52	0.51	0.43	0.38
11	0	0.14	0.20	0.32	0.28	0.34	0.36	0.40	0.45
Average deflection									0.47

Note:- (-) denotes upstream deflection

rockfill dams of modern execution and similar proportions. Its performance adds materially to confidence in the sloping impervious-membrane type of rockfill dam exemplified by the original Nantahala design.

TABLE VII
KENNEY DAM
LONGITUDINAL MOVEMENT AT CREST, IN FEET

Settlement Plate No.	Dec.12-13 1952	Nov. 20 1953	July 17 1954	Sept. 1955	June 1956	July 19 1957	July 1958	Longitudinal movement as percentage of height of fill
1	0	0.01	0.04	0.09	0.12	-0.03	0.18	0.36
2	0	0.16	0.20	0.32	0.45	0.24	0.55	0.64
3	0	0.20	0.24	0.35	0.33	0.29	0.43	0.34
4	0	0.31	0.29	0.33	0.35	N.A.	0.44	0.26
5	0	0.23	0.04	0.37	0.31	1.22	0.20	0.10
6	0	0.16	0.16	0.30	0.15	1.11	0.30	0.11
7	0	0.06	0.09	0.36	0.51	0.98	0.39	0.13
8	0	-0.07	0.00	0.12	0.18	0.83	0.04	0.02
9	0	-0.09	-0.04	0.14	0.35	0.77	-0.05	0.02
10	0	Damaged	Damaged	Damaged	Damaged	Damaged	Damaged	-
11	0	-0.13	-0.04	0.21	0.13	0.69	0.05	0.03
12	0	-0.08	-0.03	0.16	0.11	0.67	0.13	0.11
13	0	-0.08	0.02	0.14	0.20	0.75	0.26	0.36

Note:- (-) All longitudinal movements are in the direction right abutment to left abutment except where indicated by (-).

TABLE VIII
KENNEY DAM
LONGITUDINAL MOVEMENT OF BERM AT EL. 2785, IN FEET

Settlement Plate No.	June 22 1953	Nov. 20 1953	July 17 1954	Sept. 1955	June 1956	July 19 1957	July 1958	Longitudinal movement as percentage of height of fill
1	0	0.63	-	0.60	0.68	0.44	0.61	1.18
2	0	0.59	-	0.58	0.63	0.39	0.68	0.74
3	0	0.49	-	0.52	0.50	0.37	0.60	0.46
4	0	0.45	-	0.50	0.54	1.28	0.46	0.29
5	0	0.39	-	0.37	0.31	1.14	-	-
6	0	0.26	-	0.24	0.19	1.06	-	-
7	0	0.19	-	0.15	0.11	0.92	0.03	0.01
8	0	0.12	-	0.08	0.12	0.86	0.03	0.02
9	0	0.08	-	0.12	0.15	0.78	0.24	0.18
10	0	0.03	-	0.06	0.04	0.79	0.18	0.16
11	0	0.00	-	0.06	0.10	0.70	0.10	0.11

Note: All longitudinal movements are in the direction right abutment to left abutment

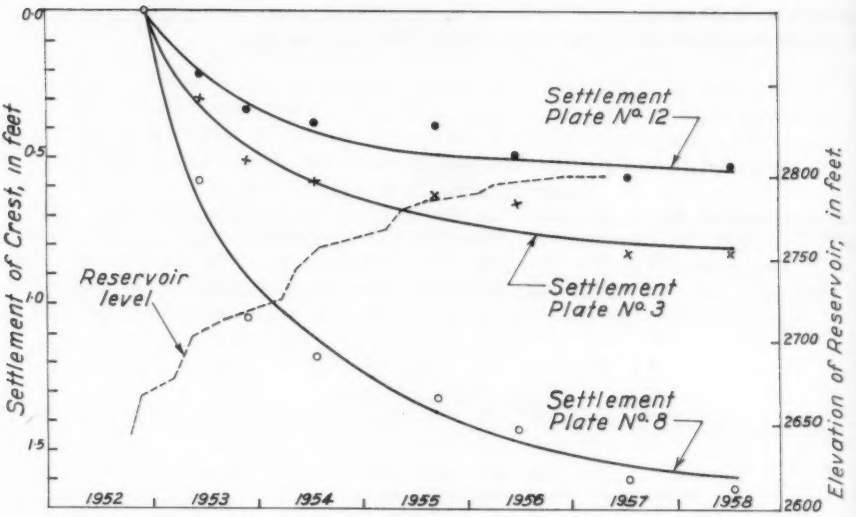
Settlement of Crest, in feet
0.0
0.5
1.0
1.5
Settlement Plate No. 12
Settlement Plate No. 3
Settlement Plate No. 8
Reservoir level
1952
1953
1954
1955
1956
1957
1958
2800
2750
2700
2650
2600
Elevation of Reservoir, in feet.

FIG. 1

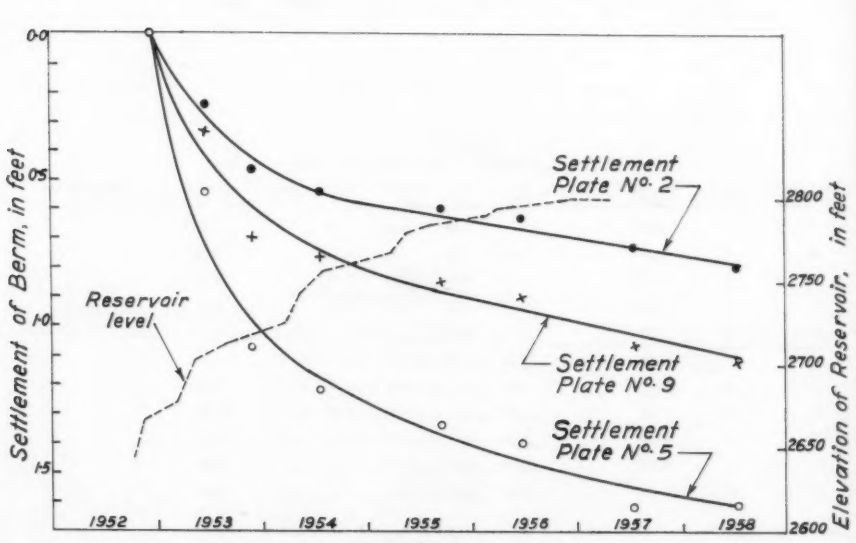
Settlement of Berm, in feet
0.0
0.5
1.0
1.5
Settlement Plate No. 2
Settlement Plate No. 9
Settlement Plate No. 5
Reservoir level
1952
1953
1954
1955
1956
1957
1958
2800
2750
2700
2650
2600
Elevation of Reservoir, in feet

FIG. 2

ROCKFILL DAMS: CHERRY VALLEY CENTRAL CORE DAM[a]

Closure by H. E. Lloyd, O. L. Moore, and W. F. Getts

H. E. LLOYD,[1] F. ASCE, O. L. MOORE,[2] M. ASCE, AND W. F. GETTS,[3] F. ASCE.—Mr. Dodd's comments on the use of a horizontal drainage blanket under the core downstream of the grout curtain in an impervious core rockfill dam is indicative of the variety of arrangements that may be used with this type of dam. The use of a horizontal drainage blanket under the downstream third of the core was considered for Cherry Valley Dam for the reasons given by Mr. Dodd, namely lowering of the phreatic line in the downstream portion of the core and the possibility of a reduction in the seepage forces acting on the downstream face. However, the core slopes finally used were such that seepage pressures on the downstream slope of the core had no appreciable effect on the theoretical stability. Of more concern to the designers was the question of assuring a tight cut-off along the line between the core and the rock foundation. The length of contact would have been much less had a horizontal drainage blanket been used.

The method of operation of the reservoir also influenced this decision. The water level will not remain at maximum elevation for more than a month or two each year. This is not a sufficient length of time to establish the phreatic line at its maximum theoretical position so that the lowering of the phreatic line is not as important as it might be otherwise. Though this was not considered in the design, it did influence the decision as to the use of a horizontal drainage blanket.

In view of these considerations, it was decided that a horizontal drainage blanket would not be used at Cherry Valley Dam. The authors are in full agreement with Mr. Dodd's statement that the advisability of a horizontal drainage blanket is dependent on the factors peculiar to each situation which must be considered in terms of cost and performance.

Mr. Lawton inquires as to the use of an earthquake allowance of 0.05 g for Cherry Valley Dam. The Cherry Valley region is not subject to violent earthquakes. No recently active faults have been found or are known to exist in the region, and there is no evidence of recent earthquake activity in the vicinity. Inasmuch as the dam is founded upon bedrock of the Sierra batholith, with its great mass of hundreds of cubic miles of dense, sound rock, there is little possibility of large earthquake forces being transmitted to the dam. For

a. Proc. Paper 1733, August, 1958, H. E. Lloyd, O. L. Moore, and W. F. Getts.

1. Mgr. and Chf. Engr., Hetch Hetchy Water Supply, Power and Utilities Engr. Bureau, Public Utilities Comm., City and County of San Francisco, Calif.
2. Constr. Engr., Hetch Hetchy Water Supply, Power and Utilities Engr. Bureau, Public Utilities Comm., City and County of San Francisco, Calif.
3. Senior Civ. Engr., Hetch Hetchy Water Supply, Power and Utilities Engr. Bureau, Public Utilities Comm., City and County of San Francisco, Calif.

these reasons seismic forces were not considered to be significant in the design of the dam, hence the use of a coefficient of 0.05 g.

Mr. Lawton also speculates as to whether or not the method in which the transition material was placed during the summer of 1954 has contributed to the differential settlement between the rockfill and the core. The writers believe not, since differential settlement between the rockfill the the core is inherent in a central core rockfill dam. The method of placement used in 1954 gave what amounted to an additional thickness of transition material between the core and the rockfill which would ease this condition somewhat. It is noted that cracking along the top of the dam has been reported at other dams of this type and is not unique to Cherry Valley Dam.

To bring the information in the original paper up to date, additional settlement and deflection data are given below.

During the 1959 run-off season construction activities on other parts of the project required that the water in the reservoir be kept below elevation 4600. The 1959 run-off in the region was below average which led to early release of the stored water so that at the present writing the reservoir has been drawn down to the lowest point since its original filling. It is interesting to note that as the reservoir was drawn down the crest of the dam moved back upstream past its position at the end of the 1958 drawdown.

As a matter of interest a set of elevations taken on the upstream rock berms is presented. Due to the large rock of which the rockfill is composed, it is rather difficult to say what the exact elevation is at any point since adjacent rocks may vary a foot or more in size and elevation. Hence the elevations recorded are an estimate of the average elevation at the points. Conclusions as to the amount of settlement at a point must keep this in mind, also the fact that during construction the same difficulties were present and the berms were constructed to an average elevation by eye rather than to an exact elevation by instrument.

An error in Plate 2 of the original paper has come to the author's attention. The upstream slope of the impervious core is 0.70 to 1 instead of 0.75 to 1 as shown on the plate.

CHERRY VALLEY DAM
SETTLEMENT AND DEFLECTION IN 1958 - 1959

Date		June 17, 1958	Aug. 14, 1958	Oct. 13, 1958	July 24, 1959
Water Surface Elevation		4686	4695	4575	4530
Sta. 4+00	deflection	0.02	0.04	-0.01	0.00
	settlement	0.10	0.10	0.11	0.10
Sta. 8+00	deflection	0.08	0.16	0.05	0.06
	settlement	0.22	0.22	0.26	0.28
Sta. 12+00	deflection	0.28	0.38	0.25	0.17
	settlement	0.37	0.36	0.43	0.49
Sta. 16+00	deflection	0.42	0.54	0.38	0.29
	settlement	0.44	0.44	0.52	0.62
Sta. 20+00	deflection	0.20	0.28	0.18	0.10
	settlement	0.36	0.37	0.43	0.50
Sta. 24+00	deflection	0.01	0.02	0.00	0.00
	settlement	0.10	0.10	0.11	0.09

TABLE I

A list of published articles on the construction of Cherry Valley Dam follows:

1. "Bids to be opened August 18 on Cherry Valley Dam Project"
   Construction Illustrated, July 1953, pg. 1
2. "Cofferdam built, contractors to rush fill placement for Cherry Valley Dam"
   Construction Illustrated, June 1954, pg. 5
3. "Cherry Valley Dam job shifts out of high"
   Engineering News-Record, August 26, 1954, pg. 30
4. "Largest hammer drills in action at Cherry Valley Dam"
   Western Construction, September 1954, pg. 57
5. "Equipment geared for big fill job"
   Construction Methods and Equipment, September 1954, pg. 50
6. "Cold storage for Sierra snow"
   Big (published by Goodyear Tire & Rubber Co., Akron 16, Ohio) volume 10, number 3, pg. 2
7. "Atkinson completing fill for Cherry Valley Dam; Spillway Outlet Works started"
   Construction Illustrated, September 1955, pg. 1
8. "High, wide and wet"
   Excavating Engineer, November 1954

CHERRY VALLEY DAM
UPSTREAM BERM ELEVATIONS
(Survey of July 24, 1959)

	4670	4640	4610	4580	4550
4+00	4570.4				
8+00	4669.5	4639.1	4609.7		
12+00	4669.0	4639.0	4608.5	4579.1	4549.8
16+00	4668.1	4637.5	4607.4	4578.4	4548.2
20+00	4669.1	4639.2	4610.0	4579.0	4550.5
24+00	4669.3	4639.4			

TABLE II

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ROCKFILL DAMS - THE DALLAS CLOSURE DAM[a]

Closure by Robert J. Pope

ROBERT J. POPE.[1]—Alternate sites for The Dalles Project were considered during the early planning stages, but rejected due to unfavorable economics. No feasable alternate method for construction of a closure dam at the selected site was devised due to the depth of river at this location.

The river bottom within the foundation area for the diversion fill was at approximate elevation -85 MSL. Rockfill obtained from excavation for the powerhouse was placed in this area as shown on fig. 6 to approximate elevation +20 during the early stages of construction. This was the portion of the fill which sustained four annual flood flows before completion of the closure without appreciable erosion. River velocities in the closure area were not measured during these floods but model studies(1) indicate the velocities during the 823,000 cubic foot per second maximum flood of 1956 to have been 30 feet per second at 5 foot depth, 32 feet per second at mid depth and 26 feet per second at 5 feet above the bottom.

The fill lift which diverted the river flow through eight skeleton units in the powerhouse was end dumped about 5 feet above the river surface or at approximate elevation 85. The height of the diversion lift was therefore about 65 feet above the surface of previously placed rock, but 170 feet above the original river bed.

Loss of rock during the diversion operation was held to a very small amount by maintaining an upstream leading edge on the 240 foot wide diversion fill. Most of the energy was thereby dissipated near the upstream edge and the smaller rock was deposited downstream but still within the width of the diversion fill. Only the finest rock was carried downstream of the dam section. The gravel and blanket materials were placed in relatively quiet water and no loss of these materials occured.

Unfortunately no method of measuring the quantity of seepage through the embankment or foundation has been devised. Considerable seepage could occur without noticeable evidence due to approximately 180 feet depth of tailwater. Underwater examination of the upstream face for silting since completion of the dam has not been made.

Reasonable agreement was obtained between sonic depth finder and lead line soundings for relatively level bottom conditions. However, when sounding to sloping or irregular surfaces, the accuracy of the sonic method decreased in proportion to the depth of water and slope of the bottom surface.

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a. Proc. Paper 1738, August, 1958, by Robert J. Pope.

1. Chf., Soils Engr. Section, Portland District Corps of Engrs., U. S. Dept. of the Army, Portland, Ore.

REFERENCES

1. Webster, Marvin J., A. M. ASCE, "The Dalles diversion made with rockfill dam," Civil Engineering, February 1957, pp 59.

HYDRAULICS OF CIRCULATING SYSTEMS[a]

Discussion by G. Dugan Johnson

G. DUGAN JOHNSON,[1] M. ASCE.—Mr. Bolieau's paper reviews the various factors involved in the design of condenser circulating water systems and describes TVA practice in connection with their major steam plants. His complete coverage of the subject makes the paper a valuable contribution to the technical literature in this field.

However, the reader must be careful to note that Mr. Bolieau's comments and conclusions are intended to apply specifically to circulating water systems for steam plant condensers and cannot be applied indiscriminately to all water pumping systems. As pointed out by Mr. Bolieau, condenser systems usually involve a rated pumping head of approximately 20 feet corresponding to friction and velocity head losses with no static lift. Although standard pumps, valves, controls, etcetera are suitable for this specialized, limited service, it should be obvious that their application to systems with higher heads (including static lift) and longer conduits must be based upon analyses of conditions requirements for each case.

As a matter of fact, even for condenser circulating water systems Mr. Bolieau's conclusions have not been adopted universally. For instance, he rules out the propeller pump which is ideally suited to handling large quantities of water under low heads (up to 40 or 50 feet) except for systems without valves, because of its high shut-off head and accompanying high brake horsepower. Not only are these pumps used by others but this objection does not apply to adjustable-blade propeller pumps(1) which have been built by the writer's company for condenser water circulating service.(2)

Mr. Bolieau's conclusion that hydraulically-operated cone valves are excellent but too expensive to use for pump discharge control may be justified for the low heads and short conduits involved in this type of service, but their cost is certainly justified for more severe service, as is evidenced by their almost universal use for pumping systems with heads above 50 feet. Also, hydraulic valve operation is not only more flexible than motor operation but permits controlled valve closure after power failure by utilizing the energy stored in an accumulator or pressure tank. Normal pump discharge control consists of automatic slow opening (90 to 120 seconds) after the pump is started, and slow closure (90 to 120 seconds) shutting the pump off automatically when the valve is almost closed, thus avoiding any noticeable pressure surges, with automatic emergency closure after power failure to the pump motor somewhat faster to minimize reversal of flow and accompanied by a controlled pressure rise of 10 to 20 percent of the rated head.

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a. Proc. Paper 1946, February 1959, by Clifton W. Bolieau.

1. Chf. Hydr. Engr., Hydr. Div., Allis-Chalmers Mfg. Co., York, Pa.

This same type of control applied to hydraulically-operated butterfly valves is quite satisfactory for low head pump discharge service. As pointed out by Mr. Bolieau, butterfly valves are also becoming increasingly popular for isolating service where gate valves had been used almost exclusively.

The Shawnee Steam Plant located on the Ohio River with 57 feet variation in water level posed an interesting problem. TVA solved it by introducing artificial static head in the form of a weir in the discharge channel so that the negative head in the discharge waterbox of the condenser would not exceed 23 feet. A similar problem at the R. E. Burger Station of the Ohio Public Service Company[3] was solved by constructing an intermediate basin at the usual 20 feet elevation below the condenser and regaining most of the additional head to low river levels by utilizing it to drive hydraulic turbines direct-connected to booster pumps in the circulating water system. The writer's company has built five of these turbines, two with ratings of 550 HP and three with ratings of 935 HP under the maximum head of 40 feet. The power developed by the turbines automatically reduces the required size of the main pump motors and the input thereto.

The writer agrees heartily with Mr. Bolieau's observations concerning the difficulties encountered in attempting to measure discharge flows accurately for full size pumps. However, there is no reason to doubt the reliability of pump capacity based upon accurately conducted laboratory model tests, provided that the prototype pumps, especially the impellers, are manufactured in reasonably exact scale proportions to the model. Although prototype capacity can be predicted accurately from model tests, there is some question as to the applicability of the Moody formula for efficiency increase from model to prototype on the basis of size. The results of the forthcoming tests on the pumps at TVA's Widows Creek Steam Plant should be very interesting.

REFERENCES

1. "Comparative Characteristics of Fixed and Adjustable-Blade Axial-Flow Pumps" by J. D. Scoville, Trans. ASME 1942 Vol. 64, p 599.

2. "Adjustable-Blade Axial-Flow Pumps for Circulating Water" by L. N. Reeve and J. D. Scoville, COMBUSTION May, 1943.

3. "Circulating-Water Turbine Gives Freedom to Plant Location" by G. W. Saathoff, ELECTRICAL WORLD - March 17, 1945.

PASSAGE OF YOUNG FISH THROUGH TURBINES[a]

Discussion by W. S. McIlquham and G. Dugan Johnson

W. S. MC ILQUHAM.[1]—The author has discussed the problem of the survival of fish passing through the tortuous path of a turbine and has asked a question in relation to turbine design as follows:

> "Should the turbine be designed to operate at a faster speed and lower horsepower than one designed in accordance with current practice in order to improve cavitation characteristics?"

The question raises a significant problem in relation to turbine design quite separate from the study of the safe passage of anadromous fish through the turbine. The trend in design is for turbines of high efficiency with minimum dimensions, maximum power and high speed as well as improved cavitation characteristics.

As a preface to the question, the author has stated that the N_S and H are specified for a turbine, so that the product of RPM and HP is constant in order to satisfy the formula

$$N_S = \frac{RPM\sqrt{HP}}{H^{5/4}} .$$

First let us note that the size of the turbine or the diameter of the runner to be used to obtain the H.P. is not fixed by the above formula. In order to consider the specific problem, let us now fix the diameter to be used and design two runners with different characteristics:

(a) a low speed and high power characteristic
(b) a high speed and low power characteristic

These turbines, if operated under the same H and the same turbine setting, i.e. the same sigma, it is expected that runner (b), the high speed-low H.P. design, would show improved cavitation characteristics over runner (a), primarily on account of the lower H.P. delivered under the same H with the accompanying lower average absolute velocity. Also, it will be noted that the difference in design, although for the same N_S, will result in a different H.P. rating for each turbine as well as the different RPM.

When considering a hydro power development, it is normal that the total capacity or H.P. is fixed and hence the specified H.P. rating of each turbine. So that, in order to meet the H.P. rating with type (b) design, a larger diameter will be required than for type (a). This will also result in an increase in N_S for type (b).

The current trend in design is to increase N_S relative to the head by increasing both speed and unit H.P. without an increase in runner diameter.

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a. Proc. Paper 1947, February, 1959, by J. F. Muir.
1. Hydr. Engr. Hydr. Div., Dominion Engr. Co., Ltd., Montreal, Canada

This in turn tends to increase cavitation phenomenon unless accompanied by improved design. In recent years runners have been improved in design but the problem of obtaining the ideal or optimum flow pattern is not yet achieved. This is evidenced by the number of runners of different design which are available for operation under the same head. Also field data on the pitting of runners shows that each design has its own cavitation characteristic.

The author has made reference to certain model runner test results carried out in the D.E. W. Hydraulic Laboratory, which indicated that a reduction in cavitation phenomenon or improved sigma coefficient accompanies an increase in speed. We would now point out that while this characteristic applies to one design, it does not follow that a runner of a different design will show the same characteristics; in fact the reverse may be true, so that the specific question asked by the author is not readily answered with a straight yes or no.

However, on the hypothesis that for the two types of runner design the ideal flow pattern has been attained, the turbine of type (b) design, i.e. relatively high speed and low unit H.P. characteristics, will have less cavitation phenomenon than type (a) design when operating under the same head and sigma setting. Or again, for turbines of specified rated H.P., type (b) design would have a larger diameter than type (a) and that type (b) on that account would have less cavitation phenomenon or better cavitation characteristics and an improved sigma coefficient.

For the safest passage of anadromous fish, it is assumed here that the turbine should be operated at that H.P. at which minimum cavitation phenomenon is generated in the flow and for runners with an ideal flow pattern this would be expected at best efficiency. For Francis runners, the best efficiency is normally at less than rated H.P. and the percentage varies with the N_S. Also the sigma setting of the turbine is important.

G. DUGAN JOHNSON,[1] M. ASCE.—Professor Muir's very interesting paper summarizes the problems experienced by young anadromous fish passing through hydraulic turbines on their outward journey to the sea.

He has quoted the results of experiments at Elwha (104 feet head) and McNary (92 feet head) dams showing 100% and 89%, respectively estimated survival of fingerlings passing through the turbines. In view of all the difficulties and uncertainties involved, the writer would be inclined to allow a $\pm$ 10% tolerance on these estimates. On this basis, the reported 89% at McNary would be considered as a possible 99% or 100% survivial, as at Elwha. Then, we could conclude that young fish pass safely through turbines under heads up to approximately 100 feet.

Professor Muir also quoted experiments proving that young fish can withstand hydrostatic pressures up to 460 feet or more, indicating that they could pass safely through fairly high head turbines (at least, up to 500 feet.) Professor Muir's own experiments indicated that healthy young salmon can also withstand near-perfect vacuum conditions without any permanent ill effects.

However, his experiments also indicated that the very high positive pressures resulting from the collapse of cavitation pockets. . . . . perfect vacuum plus water vapor pressure . . . . . could kill at least half of the fish tested in his apparatus. Considering the potential mechanical damage (cavitation pitting) to turbines as a result of these collapsing vapor pockets, it is

1. Chf. Hydr. Engr., Hydr. Div., Allis-Chalmers Mfg. Co., York, Pa.

amazing that any of the fish survived this test which, therefore, indicates that a perfectly healthy salmon fingerling is almost indestructible.

Nevertheless, we cannot accept his hypothesis that these tests indicate that substantial percentages of young salmon would be killed as the result of passing through hydraulic turbines under cavitating conditions. Even in a turbine with well-developed cavitation, only a fraction of the discharge area is being "bombarded" by collapsing vapor pockets. In addition, a fingerling passing through this bombarded area is not immobilized to receive the hammer-blow of the cavitation explosion, but may escape completely from a direct hit or may merely be thrown "off course" by it. These observations, plus the fact that almost half of the fingerlings tested lived through Professor Muir's most severe tests, even though they were imprisoned in the riser of his apparatus so that there was absolutely no escape from the intense pressure following collapse of the vapor pocket, lead the writer to conclude that the percentage of young salmon likely to be killed due to passage through any large hydraulic turbine, even under cavitating conditions, would be so small as to be negligible.

Consequently, the writer is not disposed to discuss the various aspects of turbine design that could minimize the cavitation tendency. However, it should be noted that the lower allowable sigma values applicable to lower specific speed turbines, or turbines especially designed for minimum cavitation, are usually taken advantage of by higher settings with respect to tailwater. As a result, in any well-designed installation, the tendency for cavitation to occur just beyond the full load output rating is essentially independent of specific speed.

The writer recently had occasion to become intimately associated with this problem of the passage of young fish through turbines. Since the company by which he is employed manufactured the fourteen 280 inch diameter Kaplan turbines at McNary, the U. S. Army Corps of Engineers engaged us to conduct some very preliminary tests on the passage of fingerlings through the 12 inch diameter McNary model turbine in our hydraulic laboratory. The purpose of this investigation was to explore the possibility of correlating tests under controlled laboratory conditions with data on fish mortality in the prototype, with special emphasis on survival rates with and without cavitation.

Although our abbreviated program demonstrated the possibility of various types of fingerlings surviving passage through such a small turbine under prototype conditions, it was not possible to arrive at any definite conclusions regarding the effect of cavitation on survival.

Adverse factors, such as unforseen mechanical difficulties with the fish-handling equipment and the use of hatchery fish of such sizes and species as were available in chemically treated city water at relatively high temperatures, precluded any quantitative analyses of mortality due to cavitation and/or other causes.

However, the use of models to study the factors influencing the mortality of fingerlings in passing through turbines appears to be very promising. As a result of this brief introduction to the problem, we are now in a much better position to assist the U. S. Army Engineers, or any other agency, in outlining a more comprehensive program that might lead to some realistic conclusions regarding the effects of cavitation on the mortality of young fish passing through hydraulic turbines.

We are extremely anxious to cooperate in any program that shows promise of resolving the present impasse between the "fish people" and the "power

people." It is our considered opinion that the natural resources of our great rivers should, and can, be developed fully for useful power, without interfering materially in the life cycle of such important anadromous fish as the salmon. Fish ladders, or other effective devices, are required for the upstream trip of the full grown fish to the spawning beds, and the downstream trip of the small fingerlings can be made via the spillways of the dams or through the turbines with negligible mortality. Various types of attraction devices may be required to get the young fish started down the penstocks of medium to high head turbines, but we believe that the passage can be made safely.

We feel that Professor Muir has contributed a valuable approach to this extremely important problem and trust that he will not disagree too violently with our extension of his hypothesis . . . . . that young salmon can pass safely through reasonably large hydraulic turbines under non-cavitating conditions . . . . . to include fully developed cavitating conditions.

HIGH-LIFT CONSTRUCTION METHODS FOR MASS CONCRETE[a]

Discussion by Adolf A. Meyer

ADOLF A. MEYER,[1] F. ASCE.—When reviewing the development of construction methods for mass concrete over the last 30 years, we find it closely tied to a basic postulate of design. This postulate calls for control and specific limitations of the temperature drop in restrained mass concrete. These limitations were established to keep the tensil stresses, caused by temperature drop, within an allowable range; they are considered essential for the safety of the structures. Many systems of control have been advocated and applied. They make use of one or of several of the following means to reduce the temperature drop:

1) Use of concrete mixes with very low cement content.
2) Use of cement of low specific heat.
3) Surface cooling, by pouring in shallow lifts with long intervals between lifts.
4) Precooling of the mix to obtain low placing temperatures.
5) Forced cooling of the concrete in place.

All these methods have been extensively discussed. In all these discussions, the basic postulate has been taken for granted by everybody.

But side by side with this apparent unanimity of opinion, there was a consistent outsider: From time to time we read about new hydro electric projects in Ontario, constructed by a large and highly competent firm, the construction of which followed methods which apparently disregarded all concern about temperature control. The reports talked about mixes with high cement content and about lifts of unprecedented heights. In short, this firm chose to be the heretic in the mass concrete field.

The author of this paper is to be congratulated for presenting a clear, frank and extensive description of the high-lift construction method, as practised by the Hydro-Electric Power Commission of Ontario. He explains the factors which favor its application in Ontario. He gives a most interesting discussion about the effects of this method on the quality of the structures.

The writer is particularly impressed with two aspects of the observations presented by the author.

First, these observations show that, for the methods used, the temperature drops are in the range where conventional design methods would expect them to be. They range between 55°F and 85°F.

Second, the devastating results which these high temperature drops would presage, did not occur.

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a. Proc. Paper 2061, June 1959, by Otto Holden

1. Cons. Engr., Annapolis, Md.

The author offers some explanations for these unexpected results. He points to the fact, that stresses from temperature change develop only in zones where the volume change is prevented or restrained by an outside force. Normally, such restraint exists only at or near rock foundations. Both his field explorations and the photo elastic model indicate that the influence of restraint, with increasing distance from the restraining surface, decreases much faster than is commonly assumed. Obviously only a small fraction of the total volume of a dam structure is exposed to stresses caused by temperature drop.

Of particular interest is the observation that there is no tendency for cracks to extend beyond the zone of deficient tensile strength. In shallow lift construction, cracks once started have a way of appearing in lift after lift. The secondary restraints, introduced by a slow pouring rate, must account for weaknesses not encountered with high-lift construction.

The author stresses the point that the method has been used for structures of modest heights only. The average height of the dams listed in Table 1 is 122', the highest dam there listed is 225' high. The author points to the great reserve strength which a low gravity dam undoubtedly possesses. This reserve decreases rapidly, as the height of the dam increases. It therefore can be expected that the author himself would put a limit in height on dams built by the described method. Would he care to let us know where his experience would place this limit?

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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c. Discussion of several papers, grouped by divisions.

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